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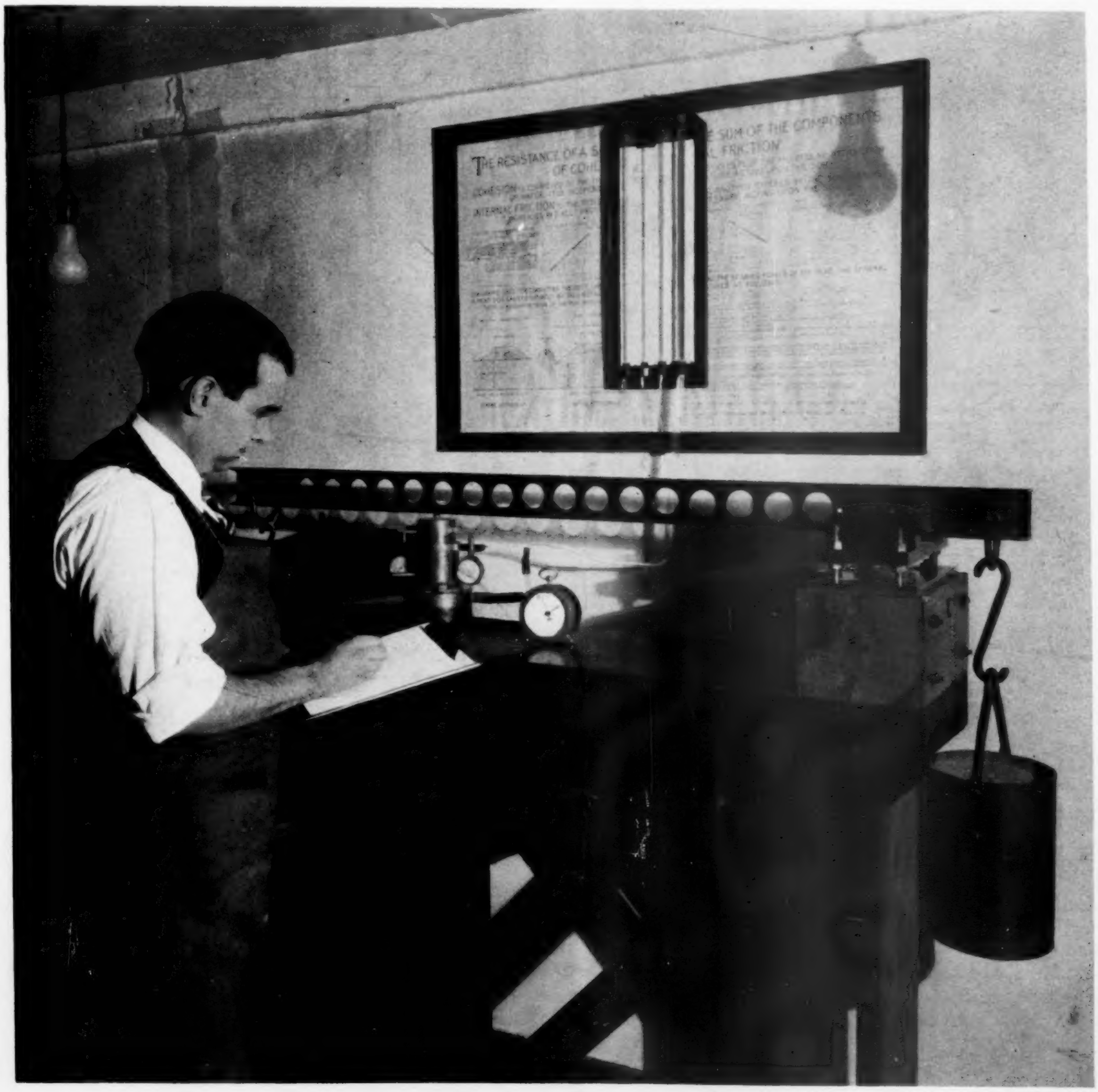

PERFORMING THE SOIL COMPRESSION TEST

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In This Issue

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A METHOD OF PREDICTING SETTLEMENT OF FILLS PLACED ON MUCK BEDS

DATA FROM COMPRESSION TEST USED IN ESTIMATING RATE AND AMOUNT OF SETTLEMENT

Reported by F. A. ROBESON, Junior Highway Engineer, Division of Tests[1]

SETTLEMENT of fills on muck soils is one of the oldest problems with which engineers have had to deal. Until recently the only guides in predicting settlement were records of observed settlements of fills thought to have been placed under conditions similar to those affecting the fill for which estimates were being made. Only very limited records are available, and for most of the older fills that have settled the amount of settlement cannot be ascertained. In railroad valuation work in the swamps in the Southeastern States there was considerable discussion as to the amount to be allowed for earthwork that could not be measured readily because of fill settlement.

Many fills must be placed on muck beds of sedimentary origin that are below the normal water level. Samples of the muck may contain considerably more moisture, by weight, than solid material. The beds range from a few feet up to 30 feet or more in thickness and generally rest upon sandy or gravelly material of relatively stable character.

It has been observed that when fill material is placed on muck with distinctly fluid characteristics, the weight of the fill causes muck to flow outward and upward at the sides of the fill. Mounds of muck are formed that tend to produce a condition of equilibrium. At the same time the weight of the fill on the muck layer forces contained water to escape by seepage through the muck and the layers of material above and below it. If these layers are porous, final settlement will be attained more quickly than when the escaping water encounters considerable resistance. Seepage of water and fill settlement continue until a condition of equilibrium between muck stability and weight of fill is reached—generally a matter of several years.

Knowledge of these characteristics has been used to improve methods of fill construction. Fills have been constructed by first placing a windrow of material along each edge of the proposed fill so as to trap the top layer of fluid muck and partly prevent the escape of muck to the sides. Fills are also placed in layers so as to allow time for the muck to consolidate under moderate load rather than subject it immediately to a heavy pressure that forces the flow of large quantities of muck from beneath the fill. When possible, granular material is used for the lower portion of the fill to facilitate escape of water pressed from the muck by the weight of the fill.

No satisfactory method for determining fill settlement in advance of construction has been developed. Studies made in connection with the construction of a hydraulic fill on muck soil at Four Mile Run on the Mount Vernon Memorial Highway have resulted in the development of methods of estimating settlements based on tests of undisturbed muck samples and data on the weight of the fill and depth of the muck deposit.

[1] This report supplements the report entitled "A Study of Hydraulic Fill Settlement", by Henry Aaron, PUBLIC ROADS, vol. 15, no. 1, March 1934.

FILL AT FOUR MILE RUN IMMEDIATELY AFTER CONSTRUCTION. PICTURE TAKEN AT LOW TIDE.

Comparison has been made between the actual performance of soft undersoils under the particular conditions existing at Four Mile Run with the performance suggested by laboratory and mathematical analysis. Settlement of the fill and the moisture contents of the soil beneath the fill, as computed from the results of tests on samples of undisturbed undersoil, are compared in the following pages with actual settlements measured in the field and with moisture contents of samples taken from beneath the fill.

In constructing the fill, gravel was pumped from the river bed by the hydraulic method and placed on a layer of soft river-bottom sediment, 10 to 30 feet thick, resting on a firm sand foundation.

The general plan of the improvement, the longitudinal profile of the fill material, and the manner in which it was deposited are shown in figure 1. Figure 2 shows cross sections of the fill.

SAMPLE BORINGS TESTED IN THE LABORATORY

Because of slides and settlement during construction, the quantity of fill material placed was considerably greater than was originally estimated to be needed. Six months after completion of the fill, borings were made through the fill material and muck undersoil as part of an investigation of the factors responsible for the necessity of using excess fill material.

The borings were of two kinds: Probe borings, marked "P", in figure 1, that disclosed only the general character of the material penetrated; and sample borings, marked "S", that furnished samples of the soft undersoil in the undisturbed natural state in addition to information on the general character of the material penetrated.

Data obtained from laboratory tests made it possible to estimate (1) the ultimate settlement of the fill; (2) the settlement that may be attained during any time interval after construction; (3) total settlement after any period of time due both to lateral flow of the soft

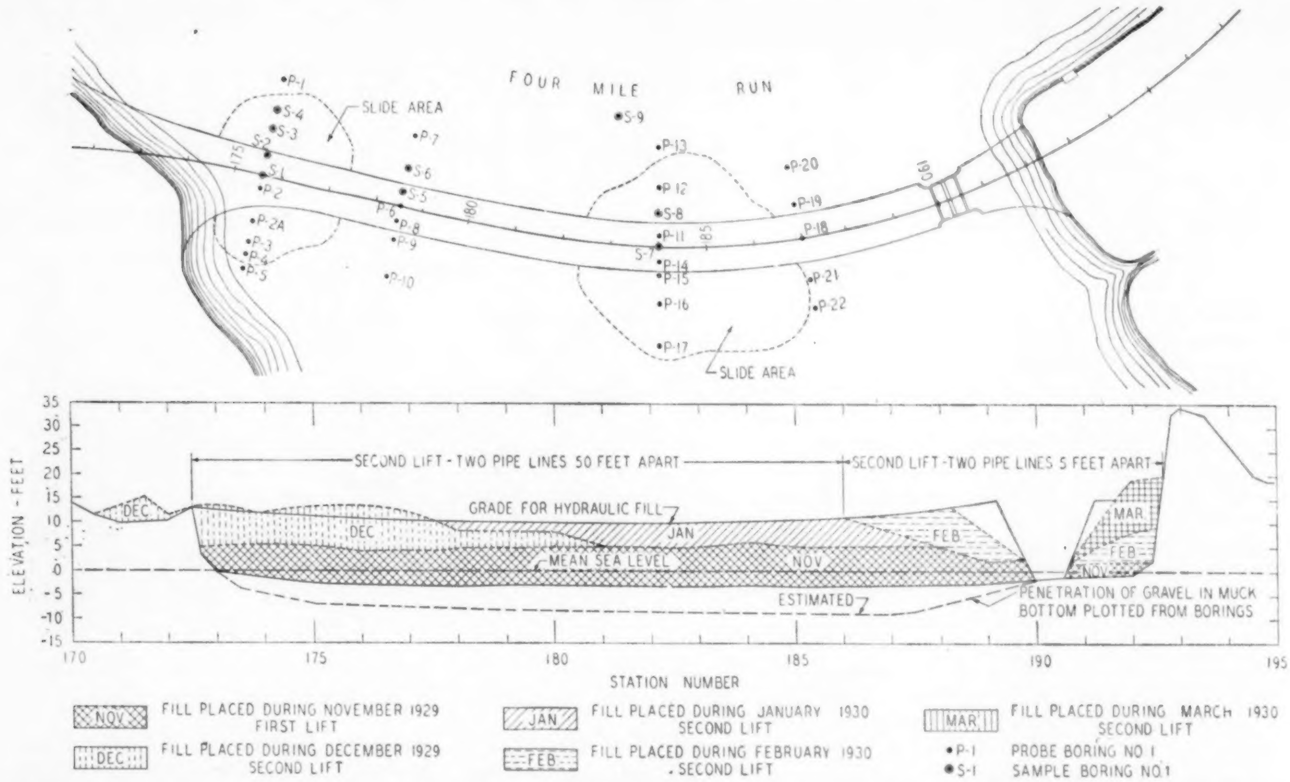

FIGURE 1.—LOCATION OF BORINGS AND PROGRESS IN CONSTRUCTING HYDRAULIC FILL AT FOUR MILE RUN, MOUNT VERNON MEMORIAL HIGHWAY.

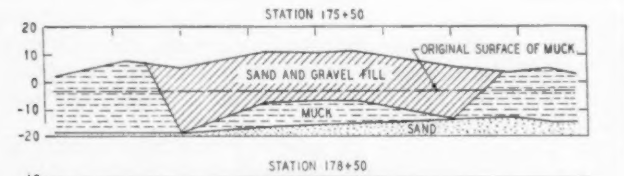

FIGURE 2.—CROSS SECTIONS OF FILL AS INDICATED BY BORINGS.

undersoil during the construction and to subsequent consolidation of the undersoil; and (4) the degree of consolidation as indicated by the moisture content, at any given depth in the undersoil at any time subsequent to construction of the fill.

The laboratory examinations included determinations of the physical characteristics of the constituents of the undersoil by the plasticity and shrinkage tests, and of the load-deformation relations of the undersoil in both undisturbed and remolded states by means of the Terzaghi compression test.

In all, 10 core samples were tested for natural moisture contents and plasticity and shrinkage constants. The results are shown in table 1. It should be first noted (table 1) that the agreement of osberved natural moisture contents with the liquid limits is generally good, further substantiating the findings of Dr. Terzaghi.[2]

TABLE 1.—*Results of laboratory tests on cores*

Boring no.	Core no.	Moisture content, natural state	Liquid limit	Plasticity index	Shrinkage limit
S-2	2	76	77	38	26
S-4	1	108	103	49	33
S-5	[1] 3	120	128	66	29
	[1] 4	107	131	64	34
S-6	5	125	132	69	32
S-7	[1] 8	64	89	48	27
	9	105	96	47	34
	10	116	112	61	32
S-9	6	79	62	30	29
	[1] 7	117	79	36	31

[1] These cores were nonuniform and were not suitable for compression tests. Therefore, they are not considered in the analysis of the test data.

The plasticity indexes and shrinkage limits are plotted against liquid limits in figures 3A and 3B, respectively. Soil classification criteria [3] indicate that

[2] See Simplified Soil Tests for Subgrades and Their Physical Significance, by Dr. Charles Terzaghi, PUBLIC ROADS, vol. 7, no. 8, October 1926.

[3] See Soil Constants, by C. A. Hogentogler, A. M. Wintermyer, and E. A. Willis, pt. 3, PUBLIC ROADS, vol. 12, no. 5, July 1931.

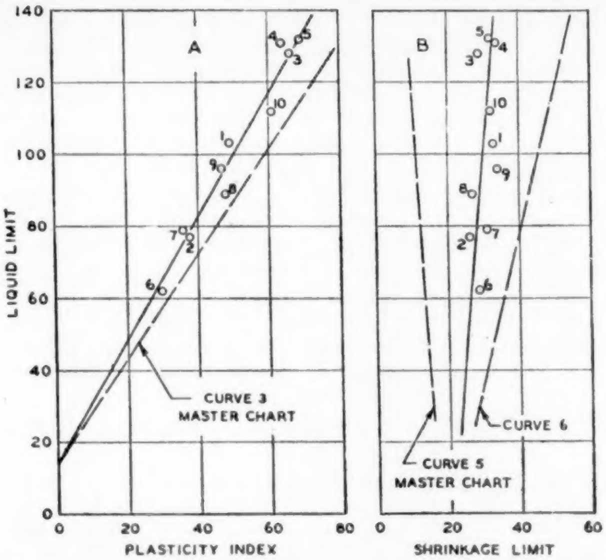

FIGURE 3.—PHYSICAL CHARACTERISTICS OF MUCK SAMPLE.

these samples have the properties of the A–8 subgrade group with less plasticity and more elasticity than the highly flocculated clays of the A–7 group.

Whereas purely inert fine clay particles could furnish the plasticity indicated by curve 3, such particles would produce shrinkage limits indicated by curve 5. Plastic colloidal clays with decomposed organic matter, however, containing elastic materials such as fibrous organic matter or diatoms, would have the constants shown in figures 3A and 3B.

The fact that the plotted plasticity indexes and shrinkage limits have approximately a straight-line relation to the liquid limits indicates variation in sand content only. Considerable change in character of fine material would cause corresponding variations from the straight-line relation.

TERZAGHI COMPRESSION DEVICE DESCRIBED

The essential features of the Terzaghi compression device[4] are illustrated in figure 4. A soil sample is placed between two porous stones, the upper one forming the base of a piston which loads the sample. Water drains freely from both faces of the sample under pressure and passes through the stones, escaping from overflow orifices *a* and *b*. This egress of water reduces the thickness of the sample which cannot be deformed laterally. The compression of the sample is measured by very sensitive micrometer dials.

Loads are applied in successive increments, each load being about double the preceding one, the maximum being equal to or greater than the pressure on the stratum from which the sample was taken. The density of the sample at various stages of the loading is indicated by either the moisture content or the ratio of pores to solids and is termed the voids ratio of the soil mass.[5] Each load increment is allowed to

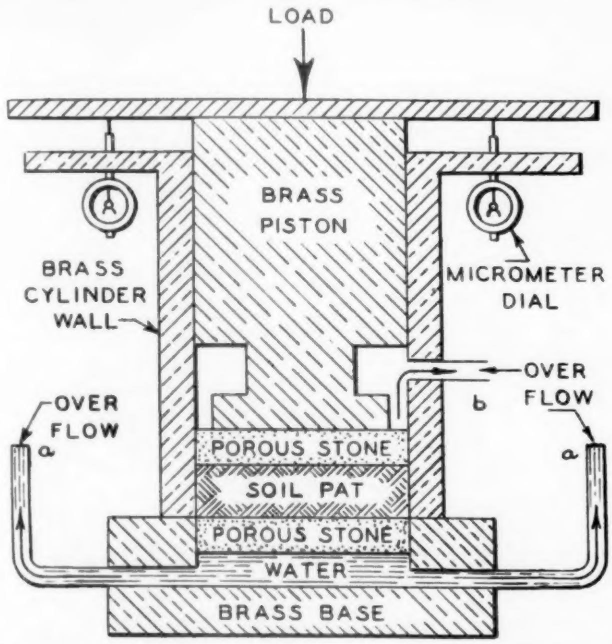

FIGURE 4.—ESSENTIAL ELEMENTS OF TERZAGHI COMPRESSION DEVICE.

act until further consolidation of the sample under that load is immeasurable. Thus the voids ratio at any stage of loading indicates the maximum density likely to be produced by the corresponding load.

In the determination of the load-deformation characteristics of the material in the natural state the samples were transferred from the hermetically sealed containers in which the soil was placed at the time of sampling to the testing apparatus with as little disturbance as possible.

A disk of undisturbed material was carefully cut from each core, trimmed to the correct diameter with a piano wire saw, and carefully placed in the testing apparatus. This transfer was made in a damp closet to prevent loss of moisture from the sample.

TEST FURNISHES LOAD-COMPRESSION, LOAD-EXPANSION, AND TIME-COMPRESSION DATA

In the test to determine properties of the disturbed material, a sample of soil from each core was first thoroughly manipulated by means of a spatula with no addition of water and then carefully placed in the testing apparatus.

The compression test furnishes three relations as follows:

1. Load-compression, or relation between voids ratio and increasing load.
2. Load-expansion, or relation between voids ratio and decreasing load.
3. Time-compression, or relation between percentage of consolidation and time.

Curves presenting these relations are shown in figure 5.[6] They were based on tests of a remolded sample from core no. 5, boring S–6, located 80 feet to the left of the centerline, station 178 + 50.

[4] See Subgrade Soil Constants, Their Significance, and Their Application in Practice, by C. A. Hogentogler, A. M. Wintermyer, and E. A. Willis, PUBLIC ROADS, vol. 12, no. 4, June 1931; Principles of Final Soil Classification, by Dr. Charles Terzaghi, PUBLIC ROADS, vol. 8, no. 3, May 1927; Soil Mechanics Research, by Dr. Glennon Gilboy, Proceedings American Society of Civil Engineers, vol. 57, no. 8, October 1931; and The Structure of Clay and Its Importance in Foundation Engineering, by Arthur Casagrande, Journal of the Boston Society of Civil Engineers, vol. 19, no. 4, April 1932.

[5] Voids ratio, e, equals moisture content, w, multiplied by the specific gravity of the soil solids, G_s.

[6] In the compression test the loads are expressed in kgs per sq. cm. One kg per sq. cm equals 2,048.5 pounds or approximately 1 ton per sq. ft. For convenience in presenting the analysis of the compression test data, therefore, kgs per sq. cm and tons per sq. ft. are used more or less interchangeably.

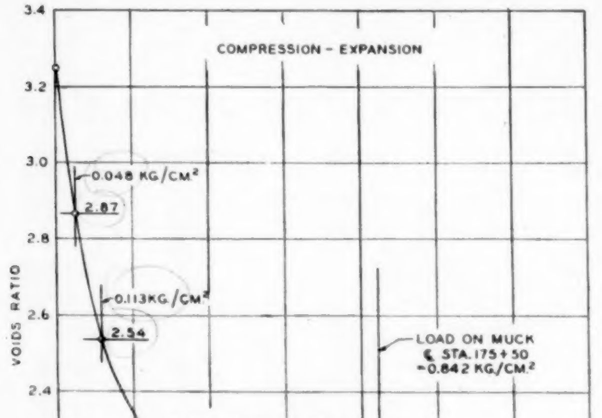

FIGURE 5.—COMPRESSION AND EXPANSION TEST RESULTS ON DISTURBED MATERIAL FROM CORE NO. 5.

According to the load-compression curve, figure 5, increasing the load from 0.048 kilogram per square centimeter to 0.113 kilogram per square centimeter compresses the sample from a voids ratio of 2.87 (moisture content 110.4 percent) to a voids ratio of 2.54 (moisture content of 97.7 percent). According to the load-expansion curve, decreasing the load from 0.4 kilogram per square centimeter to 0.1 kilogram per square centimeter increases the voids ratio from 1.91 (moisture content 73.5) to 2.03 (moisture content 78.1). According to the time compression curve, 52 percent of the above compression occurs in 25 minutes. Thus the voids ratio of the sample 25 minutes after the load was increased from 0.048 kilogram per square centimeter to 0.113 kilogram per square centimeter equals $2.87-\frac{52}{100}\times(2.87-2.54)$ or 2.70.

The relative periods of load application producing equal percentages of compression in the soil strata and in their representative laboratory samples depend upon the relative distances that water must travel to escape. If the field stratum is free to drain from both upper and lower surfaces (as when between two permeable layers) the ratio of the rate of compression of the stratum to that of the laboratory sample representing it is equal to the ratio of the squares of their respective thicknesses. If, however, the stratum can drain from only one surface then the ratio of the two rates of compression is equal to four times the ratio of the squares of their respective thicknesses. This will be developed later.

To illustrate, assume that a load of 0.048 kilogram per square centimeter that has acted for a long time on a muck layer 10 feet thick is increased to 0.113 kilogram per square centimeter. Considering that the curves, figure 5, represent the laboratory test on a sample of the field layer, the voids ratio of the field layer will decrease from 2.87 to 2.54 ultimately under this new load. This represents a decrease in thickness of $\frac{2.87-2.54}{1+2.87}\times 100$ or 8.5 percent of the total. Consequently the ultimate thickness equals $(1.00-0.085)\times 10$ or 9.15 feet, a compression of 0.85 foot. If the laboratory sample is 0.4 inch thick at a voids ratio of 2.87, the ratio of the time to effect consolidation of the stratum to the time to produce the same condition in the sample is $\frac{(10\times 12)^2}{(0.4)^2}$ or 90,000 to 1. In other words an effect is produced on the laboratory sample in 1 minute equivalent to that produced on the stratum in the field in 90,000 minutes, or about 60 days. The time-compression curve, figure 5, shows that 15 percent of the consolidation occurs in that time. Consequently the thickness of the muck layer 60 days after the increase in load was $10-\left(\frac{15}{100}\times 0.85\right)$ or about 9.9 feet.

COMPUTED SETTLEMENTS AGREE CLOSELY WITH ACTUAL SETTLEMENTS

For reasons noted in table 1, four of the core samples were considered as being unsuited for use in the settlement analysis. Compression-test data for the remaining six samples are shown in figure 6. This figure shows the profile of each boring and the compression characteristics of the materials in both the natural, or undisturbed state, and the remolded, or disturbed state.

It is noted that in every instance equal loads produced greater compression of the disturbed material than of the undisturbed material. This action, well recognized by foundation engineers, is caused by a breaking down of the soil structure by mixing.

Tables 2 and 3 and figure 7 show computed moisture contents and settlements based on the characteristics of the materials and these values are compared with actual settlements as measured and moisture contents as determined by drying samples.

TABLE 2.—*Computed and measured moisture contents*

Material from core	Moisture content	
	Computed	Measured
	Percent	*Percent*
1	123	108
2	60	76
5	123	125
6	77	79
9	86	105
10	109	116

TABLE 3.—*Estimated settlements caused by lateral flow that occurred during construction and observed* [1] *and computed settlements that occurred after construction*

	40 feet left of 175+50			At 175+50			30 feet right of 175+50			30 feet left of 178+50			At 178+50		
	Computed		Measured	Computed		Measured	Computed		Measured	Computed		Measured	Computed		Measured
	Core 2	Core 10		Core 2	Core 10		Core 2	Core 10		Core 2	Core 10		Core 2	Core 10	
	Feet	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
1 month	0.2	0.9	0.3	0.3	0.9	0.4	0.3	1.0	0.3	0.3	0.9	0.3	0.2	0.9	0.2
6 months	.6	2.2	.8	.6	2.2	.9	.6	2.2	.7	.6	2.2	.9	.5	1.8	.7
12 months	.7	2.8	1.0	.8	2.8	1.1	.8	2.8	.9	.8	2.8	1.1	.6	2.1	.9
20 months	.9	3.1	....	.9	3.1	1.4	.9	3.1	....	.9	3.1	....	.7	2.2	1.1
36 months	1.0	3.4	....	1.0	3.3	1.6	1.0	3.3	....	1.0	3.3	....	.8	2.3	1.3
Ultimate	1.6	4.0	....	1.7	3.9	....	1.6	3.8	....	1.6	3.9	....	1.0	2.6	....
Lateral flow	3.5	1.9	....	3.2	1.6	....	3.1	1.5	....	2.4	.8	....	4.4	3.1	....

	30 feet right of 178+50			20 feet left of 184+00			At 184+00			30 feet right of 184+00			At 187+00		
	Computed		Measured	Computed		Measured	Computed		Measured	Computed		Measured	Computed		Measured
	Core 2	Core 10		Core 2	Core 10		Core 2	Core 10		Core 2	Core 10		Core 2	Core 10	
	Feet	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
1 month	0.2	0.9	0.3	0.2	0.4	0.2	0.2	0.4	0.3	0.1	0.4	0.4	0.2	0.6	0.5
6 months	.5	2.1	.8	.4	1.4	.8	.4	1.5	.9	.3	1.4	1.0	.5	2.0	1.4
12 months	.7	2.6	1.1	.6	2.3	1.4	.6	2.2	1.5	.5	2.0	1.6	.7	2.8	1.9
20 months	.8	2.9	....	.7	2.9	....	.7	2.9	1.9	.6	2.5	....	.9	3.4	2.4
36 months	.9	3.1	....	1.0	3.8	....	1.0	3.7	2.9	.7	3.1	....	1.2	4.2	3.1
Ultimate	1.5	3.6	....	3.4	8.2	....	2.8	6.8	....	1.7	4.7	....	2.6	6.0	....
Lateral flow	3.6	2.0	....	2.9	1.9	....	4.9	3.8	....	16.7	15.6	....	5.3	3.8	....

[1] Measurements on centerline locations continued after construction of pavement.

Data for 10 locations are given in figure 7. The shaded bands show the range of possible settlements as computed from data obtained by tests of core 2 material representing the stiffest muck and core 10 material representing the softest muck in the deposit. Choice of these two was made on the basis of the load-compression and time-compression characteristics of the six cores investigated.

Table 3 contains the settlement data from which figure 7 was plotted. It will be noted that for the four centerline locations the measured settlements are recorded for 36 months. This is because only the levels at the centerline location were continued after the construction of the paved roadway.

According to table 3 the losses in thickness resulting from lateral flow were, in many cases, much greater than the computed ultimate settlements. This indicates that these losses are likely to be proportionately greater than those resulting from settlement in fills over similar muck beds.

All of the observed settlements are within the range of the computed maximum and minimum settlements. Since the computed maximum settlements were always greater than the observed settlements, errors in the computed settlements are on the conservative side. The differences between observed and computed settlements and moisture contents are not due to faulty theory but result from certain assumptions used in the computations in the absence of information that could only be obtained with considerable difficulty.

ASSUMPTION AS TO CHARACTER OF MUCK MUST BE MADE WITH CARE

In the foregoing computations there were two assumptions that are most likely to be at fault. They are that the muck was uniform throughout the depth of the deposit at any location and that moisture was free to flow from both top and bottom of the compressed layer. Actually, boring S–7, from which three cores were taken, indicated mucks having liquid limits varying from 89 to 112, and boring S–9, from which two cores were taken, had a liquid limit range of 62 to 79.

Visual inspection of the cores showed that the sand content varied considerably within the depth of the cores themselves. This was positive evidence of nonuniformity of the soil within a 1-foot range.

As previously stated, the rate of settlement is but one-fourth as rapid when the water is free to flow from only one face instead of both faces of the soil layer. The intrusion of stiff soil laminations into a more permeable layer tends to interfere with the free flow of water and to retard the process of settlement. In a layer of stiff muck, thin deposits of soft muck tend to increase the rate of settlement. Therefore, assuming free flow through both top and bottom of the layer, settlements computed on the basis of the stiff muck are apt to be less than those observed, whereas settlements computed on the basis of the soft muck will undoubtedly be larger than those actually occurring.

This is illustrated in figure 8. All the information in the case of boring S–2 indicates that muck represented by core 2 dominates. As is shown by figure 8, the observed settlement is slightly greater than that computed on the basis of core 2 material.

All the information in the case of boring S–7 indicates that core 10 material dominates but the deposit contains layers of core 9 material. As would be expected, therefore, the observed settlement (full-line curve, fig. 8) lies between settlements computed on the basis of the materials from cores 9 and 10, on the assumption of free flow in both directions.

The observed settlements, however, are greater than those computed on the basis of core 10 material when free to discharge water from one face only. It is

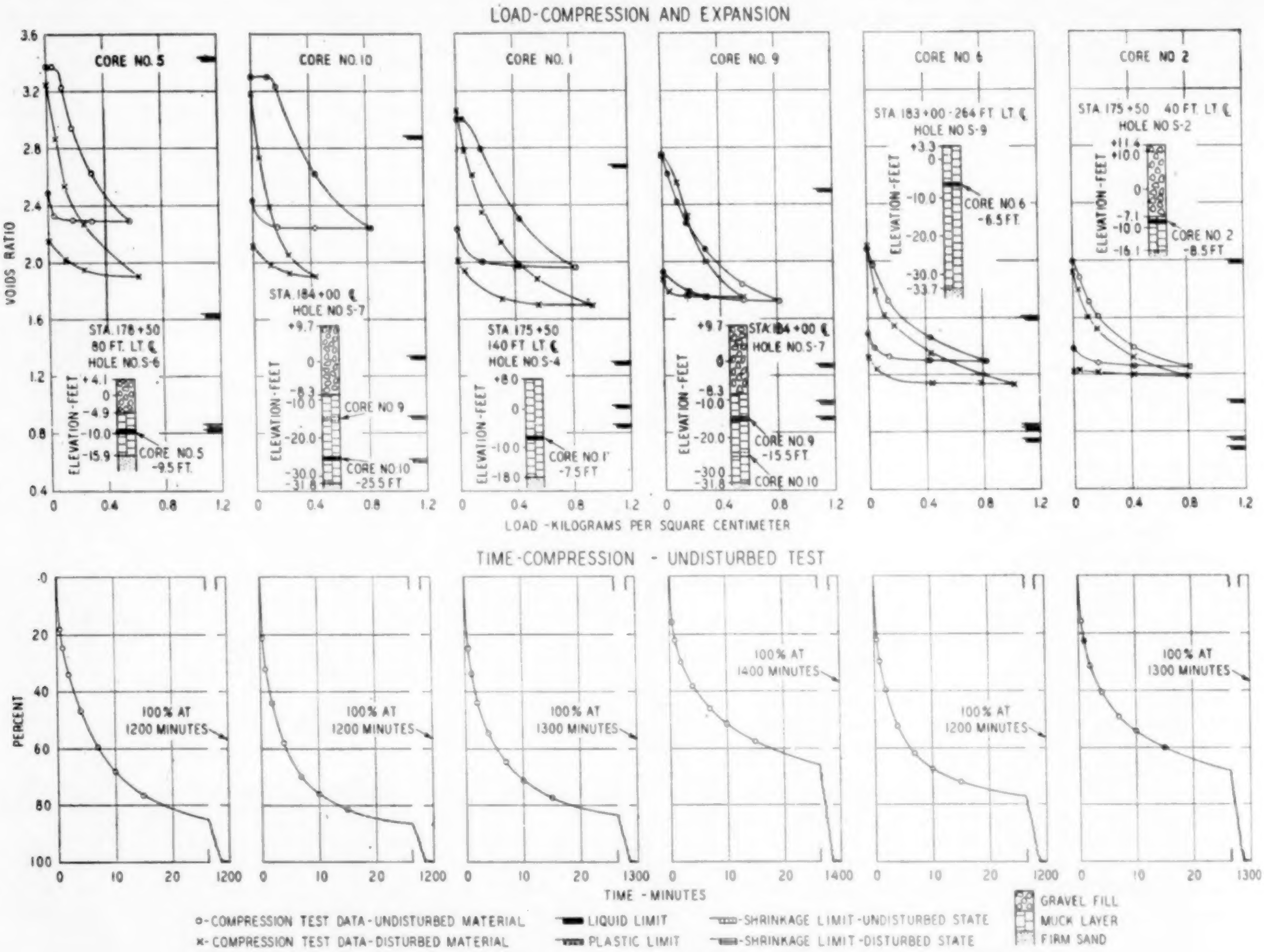

FIGURE 6.—ESSENTIAL DATA ON CORE MATERIALS USED IN THE ANALYSES.

indicated that the laminations of core 9 material retard the flow of water through the core 10 material but do not completely seal off the flow from either face.

In this connection it can be noted in table 2 that there is a much better agreement between the computed and measured moisture contents of core 10 material than of core 9 material.

RESULTS OF INVESTIGATION SUMMARIZED

The study of conditions at Four Mile Run has resulted in the following conclusions:

1. The muck at Four Mile Run belongs to the truly unstable group designated as A–8. It is distinguished by high water capacity, relatively low plasticity, and high elasticity.

2. The wide range of observed moisture contents—76 to 125 percent of weight of dried soil—is caused by differences in the compressibility of the soil at different locations in the deposit.

3. Differences in the compressibility of the soil result from variations in the clay content more than from variation in the activity of the clay content.

4. The agreement between the computed and the observed settlements indicates that the settlements of soil layers in place follow the same laws used in the analysis of data obtained by laboratory compression tests. The lack of agreement between observed and computed settlements is caused by lack of uniformity of the soil deposits. The theory regarding the relative amounts and rates of consolidation of laboratory samples and field strata seems entirely sound.

5. The agreement between the computed and the determined moisture contents, although not exact, is close enough to emphasize that wide variation in observed moisture contents at different locations in a soil deposit are not the result of haphazard circumstances, but instead are the demonstration of fairly well understood physical laws that control the consolidation of sediments.

6. Close agreement between observed and computed settlements is due largely to the uniformity of the soil deposit investigated. The greater the uniformity the more accurately will the computed settlements indicate the actual settlements.

7. Lateral flow is a major cause for concern in placing fills on muck deposits of the type described. Construction methods that will lessen or prevent lateral flow are highly desirable and contribute to greater accuracy in forecasting probable settlements.[7]

TEST PROCEDURE AND ANALYSIS OF DATA DISCUSSED

The foregoing discussion has been prepared to illustrate in a general way how problems of fill settlement

[7] See A Study of Hydraulic Fill Settlement, by Henry Aaron, PUBLIC ROADS, vol. 15, no. 1, March 1934.

TABLE 4.—*Data furnished by the combined compression and permeability tests on Four Mile Run core sample no. 5 remolded at natural moisture content* [1]

A—GENERAL DATA

Depth of soil chamber, start of test	0.4254 inches.
Cross-sectional area of compressibility chamber, A	38.5 sq. cms.
Lever ratio (loads placed directly on piston)	1:1.
Cross-sectional area of permeameter tube, a	0.10 sq. cms.
Specific gravity of soil solids, G_s	2.60.
Initial moisture content	123.0 percent.
Weight of water at end of test	21.05 grams.
Weight of dry soil at end of test	25.46 grams.
Moisture content at end of test	82.7 percent.

B—LOAD-COMPRESSION AND TIME-COMPRESSION DATA

Date	Hour	Elapsed time (t)	Load		Micrometer reading			Consolidation	
			Increment	Total unit pressure (p)	Left	Right	Average		
	p. m.	*Minutes*		*Kg per sq. cm*	*Inches $\times 10^{-4}$*	*Inches $\times 10^{-4}$*	*Inches $\times 10^{-4}$*	*Inches $\times 10^{-4}$*	*Percent*
Aug. 28	2:20	------	None	None	0	0	0	------	------
Do	2:20	0	First	0.048	0	0	------	------	------
Aug. 29	1:30	1,390	do	.048	379	379	379	------	------
Do	3:30	0	Second	.113	379	379	379	0	0
Do	3:30.5	.5	do	.113	418	416	417	38	11.6
Do	3:31	1	do	.113	429	427	428	49	15.0
Do	3:32	2	do	.113	444	443	443.5	64.5	19.7
Do	3:33	3	do	.113	455	453	454.5	75.5	23.1
Do	3:34	4	do	.113	464	463	463.5	84.5	25.8
Do	3:35	5	do	.113	470	472	471	92	28.1
Do	3:36	6	do	.113	478	480	479	100	30.5
Do	3:38	8	do	.113	489	491	490	111	33.9
Do	3:40	10	do	.113	499	501	500	121	37.0
Do	3:42	12	do	.113	504	508	506	127	38.8
Do	3:45	15	do	.113	516	513	514.5	135.5	41.4
Do	3:50	20	do	.113	530	535	532.5	153.5	46.9
Do	3:55	25	do	.113	547	550	548.5	169.5	51.8
Do	4:00	30	do	.113	559	562	560.5	181.5	55.4
Do	4:15	45	do	.113	586	588	587	208	63.5
	a. m.								
Aug. 30	9:10	1,060	do	.113	689	697	693	314	95.9
	p. m.								
Do	1:25	1,315	do	.113	700	713	706.5	[2] 327.5	100.0
Do	3:30	0	Third	.244	700	713	------	------	------
Sept. 1	2:30	2,820	do	.244	968	988	978	------	------
Do	4:35	0	Fourth	.634	968	988	------	------	------
Sept. 2	1:10	1,235	do	.634	1,329	1,365	1,347	------	------

C—LOAD-EXPANSION DATA

	p. m.								
Sept. 2	3:25	------	Third	0.244	1,329	1,365	------	------	------
	a. m.								
Sept. 3	11:45	------	do	.244	1,284	1,314	1,299	------	------
Do	11:45	------	Second	.113	1,284	1,314	------	------	------
Sept. 4	9:10	------	do	.113	1,218	1,250	1,234	------	------
Do	9:10	------	First	.048	1,218	1,250	------	------	------
Sept. 5	9:30	------	do	.048	1,114	1,144	1,129	------	------

D—PERMEABILITY TEST DATA [3]

Date	Temperature (T)	Elapsed time	Permeameter tube reading	Head of water [4]	
				h_1	h_2
	°C.	*Minutes*		*Cm*	*Cm*
Aug. 30	27.2	0	0.11	24.14	------
		60	.26	------	22.89
Do	27.2	0	.11	24.14	------
	------	60	.25	------	22.99
Average	------	------	------	24.14	22.94

[1] Abbreviated to include only one set of time-compression readings and one permeability test.
[2] Total consolidation under second load increment.
[3] For test performed at end of second load increment.
[4] Values corrected for capillary rise in permeameter tube.

may be attacked. The soil specialist dealing with such problems will desire a more detailed account of the procedure involved. In the following pages an effort is made to give such information.

The small number of laboratories equipped to make the compression test follow a more or less standard procedure but no detailed description of the procedure is known to have been published. It is not the primary purpose of this report to give the procedure, but the condensed data with computations and explanations presented in table 4 will enable one to perform the test.

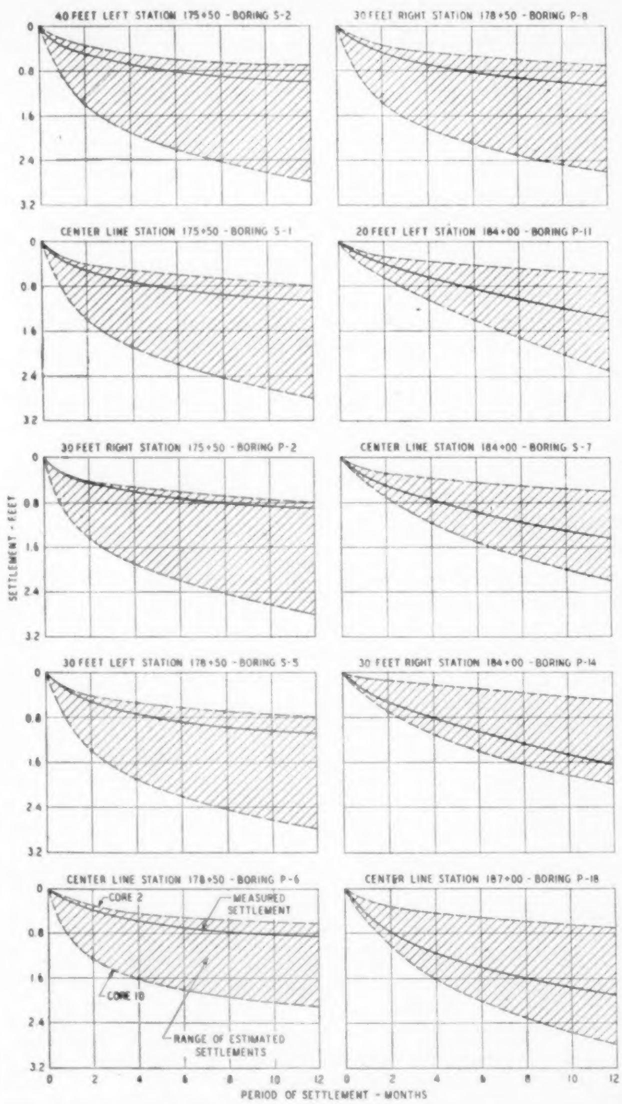

FIGURE 7.—COMPARISON OF MEASURED AND ESTIMATED FILL SETTLEMENTS.

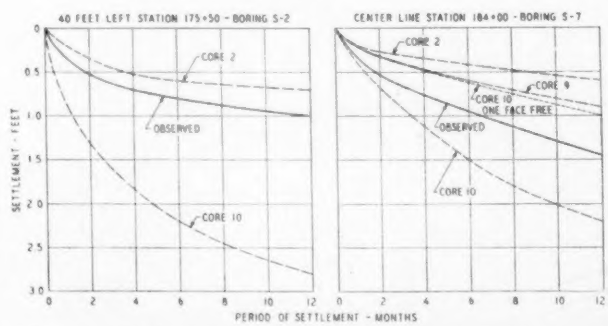

FIGURE 8.—COMPUTED AND MEASURED SETTLEMENTS OF RELATIVELY STIFF AND SOFT MUCKS.

Table 4 shows the data from which the curves of figure 5 were drawn. It also shows the results of one permeability test. The permeability test is not used in the subsequent analyses and is presented in table 4 only because it is usually performed in conjunction with the compression test. In performing the permeability test a permeameter tube is attached to one of the orifices a, figure 4, with the other orifice a closed, permitting the flow of water through the soil under known heads, the water escaping through orifice b.

Table 4–A presents such general information concerning the sample and apparatus as is necessary in computing results. If routine tests are not contemplated, it is necessary to determine the specific gravity by a separate test.

Table 4–B presents data taken during the application of load to the sample and includes a complete set of time-displacement readings. At the start of the test a pin holds the piston in a fixed position in the cylinder. Dial-type micrometers are attached and set at a suitable initial reading, which was zero in this case.

Table 4–C presents the load-expansion data. The loads are usually removed in reverse order to that used in the load-compression portion of the test.

Table 4–D gives the permeability test data. This test is usually run after the application of each load increment when the soil has reached a constant voids ratio for that load. The drop in head as measured in a permeameter tube is noted for a 60-minute interval. A check run is usually made to make sure that the apparatus is working properly as leaky apparatus must be guarded against.

Table 5 shows the computed results of the load-compression and load-expansion tests. The loads and average micrometer readings are taken from table 4. The thickness of the soil cake can be determined for all loads by subtracting the average micrometer reading from the initial depth of the soil chamber, which is 0.4254 inch. If the initial micrometer reading is other than zero, it is necessary to apply a correction to each average reading before the actual thickness of the soil cake can be determined.

TABLE 5.—*Results of combined compression and permeability test on core sample no. 5, remolded at natural moisture content*

LOAD-COMPRESSION DATA

Load		Average micrometer reading	Thickness of soil cake (d)	Voids ratio (e)	Temperature (T)	Coefficient of permeability	
Increment	Total pressure (p)					For T° C. (K_T)	For 20° C. (K_{20})
	Kg per sq. cm	*Inches* $\times 10^{-4}$	*Inches* $\times 10^{-4}$		*° C.*	*Cm/sec.* $\times 10^{-6}$	*Cm/sec.* $\times 10^{-6}$
None	0	0	4,254	3.25			
First	0.048	379	3,875	2.87			
Second	.113	707	3,547	2.54	27.2	0.033	0.028
Third	.244	978	3,276	2.27			
Fourth	.634	1,347	2,907	1.90			
LOAD-EXPANSION DATA							
Third	0.244	1,299	2,955	1.95			
Second	.113	1,234	3,020	2.02			
First	.048	1,129	3,125	2.12			
None	0		[1] 3,155	2.15			

[1] Computed value.

CHANGES MADE IN NOMENCLATURE

The nomenclature used in this report differs in some respects from that previously published. Changes were made for the purpose of clarity. The nomenclature is as follows:

e = voids ratio.
d = thickness of soil sample at e voids ratio.
d_0 = thickness of soil sample at zero voids ratio.
W_s = weight of dry soil.
G_s = specific gravity of soil solids.
A = cross-sectional area of compressibility chamber.
k = coefficient of permeability (general designation).
k_T = coefficient of permeability at temperature T° C.
a = cross-sectional area of permeameter tube.
$t_2 - t_1$ = elapsed time for permeability measurement.
h_1 and h_2 = the heads of water observed at times t_1 and t_2, respectively.
n_T and n_{20} = viscosities of water at T° C. and 20° C., respectively.
k_{20} = coefficient of permeability corrected to 20° C.
p = pressure per unit area.
B = intercept of straight-line portion of the semilog plot of the compression curve with the 1.0 kilogram per square centimeter ordinate.
Z = slope of the compression curve when plotted on a semilog scale or change in value e for unit change of log p.
$x = 2 + \log p$.
e_1 = voids ratio under pressure p_1.
e_2 = voids ratio under pressure p_2.
d_1 = thickness of soil sample at e_1.
D_1 = thickness of field layer at e_1.
t = time, see development of formula 7, p. 261.
t_d = time required for soil sample to consolidate from e_1 to e_2.
t_D = time required for field layer to consolidate from e_1 to e_2.
IW_s = weight of immersed solid material.
V = volume of soil mass.
G_w = specific gravity of water.

The voids ratio, e, can be computed for any thickness of soil cake, d, by the formula

$$e = \frac{d}{d_0} - 1 \qquad (1)$$

The value of d_0 represents the thickness of the soil cake at zero voids ratio and is constant for any single test. It is determined from the weight of dry soil, W_s, specific gravity of soil solids, G_s, and area of soil chamber A by the formula

$$d_0 = \frac{W_s}{G_s \times A} \qquad (2)$$

Substituting the values given in table 4 for these factors for this particular test

$$d_0 = \frac{25.46}{2.60 \times 38.5} = 0.2545 \text{ centimeter or } 0.1002 \text{ inch.}$$

It is noted from table 5 that the thickness of the soil cake for the zero load of the rebound is given. Since

the weight of the piston is acting on the sample as long as the micrometers are in position, this value must be computed. Usually, for stiff soils the load of the piston at the final micrometer reading is considered as zero, but in soft muck soils this assumption results in an error of measurable proportions. This thickness is computed as follows:

According to the data in table 4–A, the amount of water in the soil at the end of the test is 21.05 grams. If placed separately in the soil chamber this water will occupy a thickness of

$$\frac{21.05}{38.5\times 2.54}=0.2153 \text{ inch.}$$

Since the soil has a thickness of 0.1002 inch, the total thickness of soil cake for zero load should be 0.2153+0.1002, or 0.3155 inch. This method is sometimes used to check the results on stiff soils.

The time-compression relation is shown for the second load increment in table 4–B. The two micrometer readings were averaged and the total deformation under this load found to be 327.5×10^{-4} inches. Considering this value as 100 percent compression the corresponding percentage for all time readings are computed as shown.

The coefficient of permeability, k, is the velocity of flow in centimeters per second, feet per second, etc., at a hydraulic gradient of unity, that is, when the drop of pressure between two points in a soil column equals the horizontal distance between the points. The coefficient of permeability is dependent on temperature and, accordingly, the practice has been to correct all determined values of k to 20° C. The general formula [8] for computing the coefficient of permeability k_T is

$$k_T=\frac{2.3\,ad}{A(t_2-t_1)}\log\frac{h_1}{h_2} \qquad (3)$$

in which

k_T = coefficient of permeability at T°C.
a = cross-sectional area of the permeameter tube.
d = the thickness of the soil cake.
A = cross-sectional area of the soil cake or compressibility chamber.
t_2-t_1 = time in seconds.
h_1 and h_2 = heads of water acting at times t_1 and t_2, respectively.

Substitution of the values given in tables 4 and 5 gives,

$$k_T=\frac{2.3\times 0.10\times(0.3547\times 2.54)}{38.5\times 3{,}600}\log\frac{24.14}{22.94} \text{ or}$$

$$k_T=0.033\times 10^{-6} \text{ centimeters per second.}$$

The temperature of air and water during the test was 27.2° C. The correction coefficient for temperature equals $\frac{n_T}{n_{20}}$ in which n_T and n_{20} are values of the viscosity of water at T° C. and 20° C., respectively. For T=27.2° C. the correction coefficient becomes 0.844. Consequently, $k_{20}=k_T\times 0.844=0.028\times 10^{-6}$ centimeters per second.

JUDGMENT AND EXPERIENCE NEEDED IN INTERPRETING TEST DATA

With proper equipment no difficulty should be encountered in performing the compression test or in expressing the results. In contrast, however, interpretation of the test data is a matter of judgment and experience that does not lend itself so readily to standardization. The basis of the engineer's judgment is his opinion of the extent to which the conditions of test represent those produced by the environment of the particular soil stratum in place.

[8] For derivation of this formula see Soil Mechanics Research, by Dr. Glennon Gilboy, Proceedings American Society of Civil Engineers, vol. 57, no. 8, October 1931.

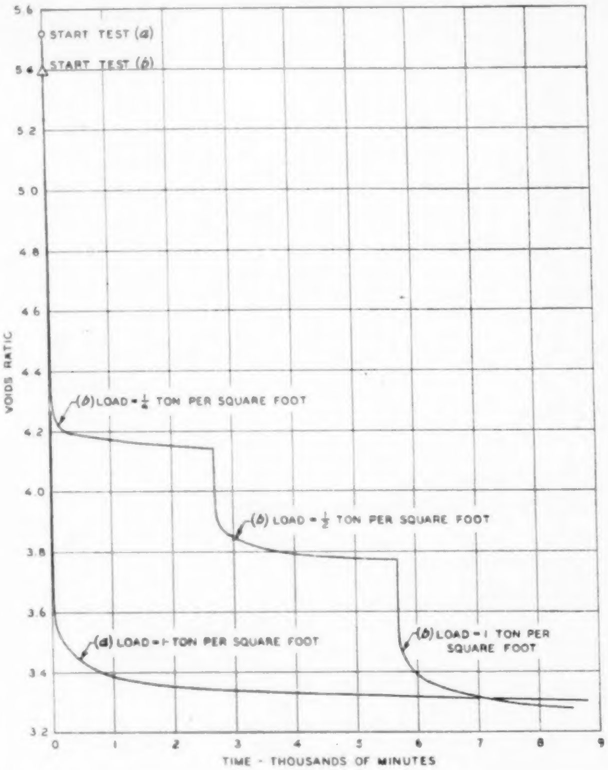

FIGURE 9.—EFFECT OF NUMBER OF LOAD INCREMENTS ON VOIDS RATIO OF MUCK SOIL.

The methods used in computing the moisture contents and the settlements are described in detail because they call attention to some of the considerations that influence the interpretation of the test data.

The following method of applying the test data has been used. Other methods might have been used, but the method described was chosen as suitable in this case, since the relative effect of the compression and sliding that occurred in the undersoil during placement of the fill could not be determined. Also, it was not known whether the core samples represented the muck as it existed before the construction of the fill, or in some condition of partial disturbance caused by the fill construction. In this analysis assumptions were made as follows:

1. The compression is direct, with no lateral flow involved after completion of the fill.

2. The muck bed had reached a state of ultimate consolidation caused by its own weight before placement of the fill and was in natural undisturbed state.

3. The weight of muck in the original layer is equivalent to that of a material having a specific gravity of 2.6 and a voids ratio as indicated by the load-compression curve.

4. Hydrostatic uplift exists below mean water level or zero elevation.

5. The effective weight of the gravel fill is 110 pounds per cubic foot above zero elevation and 68.5 pounds per cubic foot below zero elevation. The effective weight

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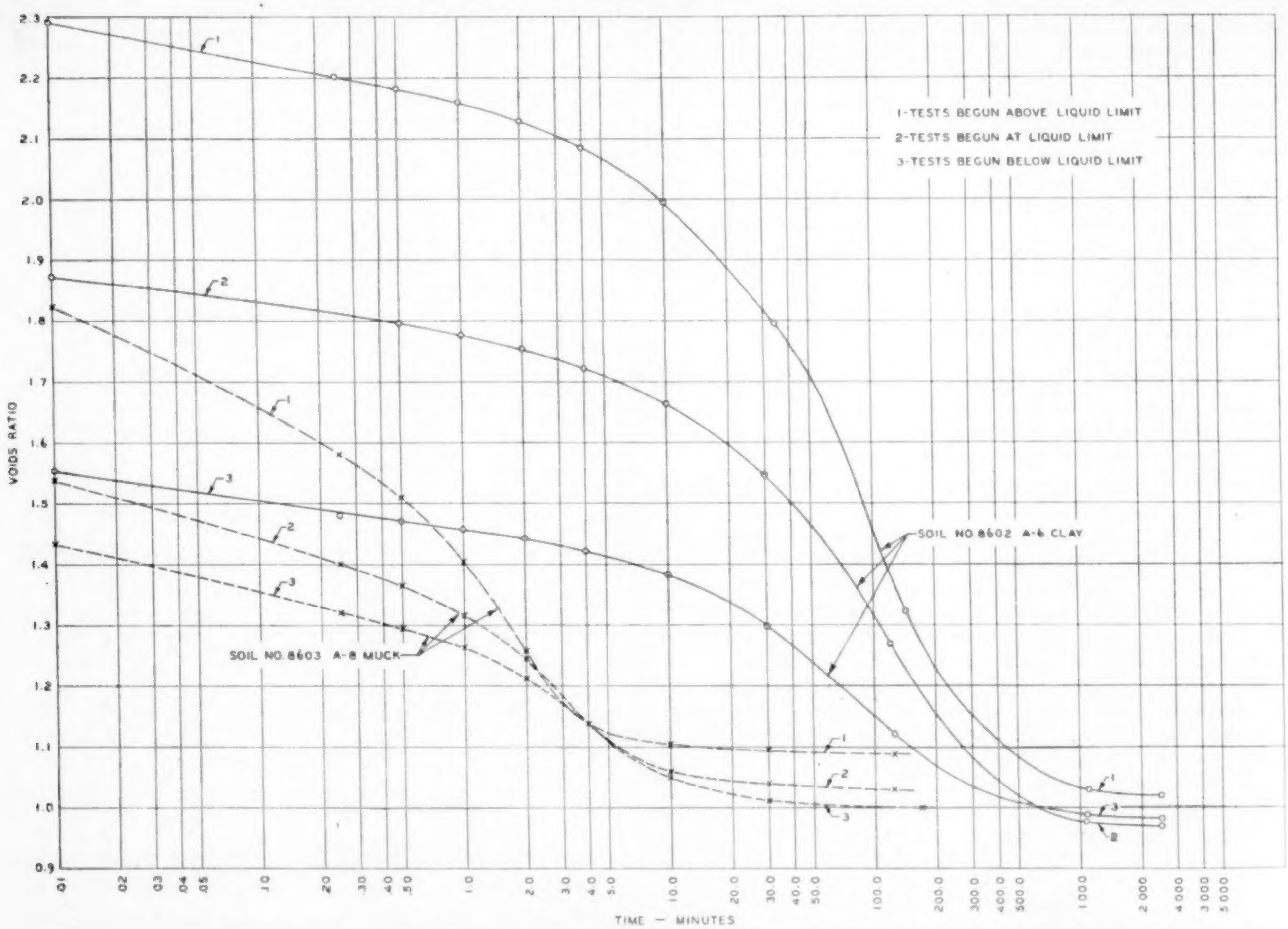

FIGURE 10.—EFFECT OF INITIAL MOISTURE CONTENT ON RESULTING VOIDS RATIO UNDER A PRESSURE OF 2 TONS PER SQUARE FOOT.

of displaced muck is 54 pounds per cubic foot above zero elevation and 33 pounds per cubic foot below zero elevation.

6. In computing the moisture contents, the loads were assumed to equal the weight of the materials above the sample location as disclosed by the test borings, and the period during which these loads were acting was assumed to be 6 months.

7. The average of the voids ratios, taken at 1-foot intervals from top to bottom of the muck layer, was used in computing the fill settlement. The actual voids ratios at the location of the particular sample, in contrast, were used in computing the moisture contents.

8. The load-compression curve discloses the relation between load and voids ratio within the load range shown, regardless of (*a*) the number of increments in which the load is applied, and (*b*) the moisture content of the muck at the start of the test, provided the pores are entirely filled with moisture. This assumption is warranted by the results of studies made in the Bureau's soils laboratory.

The results of two compression tests on a muck soil are shown in figure 9, in which voids ratios are plotted against time. In test "a", starting at a voids ratio of about 5.52, the sample was compressed under a load of 1 ton per square foot for about 6 days. The final voids ratio was about 3.30. In test "b", starting at a voids ratio of about 5.39, the sample was compressed under the same load applied in 3 increments as shown, each increment acting for about 2 days. The final voids ratio was about 3.27, varying less than 1 percent from that of test "a."

INITIAL MOISTURE CONTENT FOUND TO AFFECT ULTIMATE COMPACTION BUT SLIGHTLY

The results of two sets of compression tests started at different moisture contents are shown in figure 10. To facilitate the comparison, time is plotted on a logarithmic scale. The three initial moisture contents for each set are designated "above the liquid limit", "at the liquid limit", and "below the liquid limit." A load of 2 tons per square foot was used in each test. The two soils, an A–6 clay and A–8 muck, represent two types of material commonly encountered in settlement problems.

The variation in the resulting voids ratios of the three tests on the clay soil is the difference between 1.02 and 0.97 or 0.05, a variation of about 5 percent. Those of the muck show a variation in voids ratios of 0.09, or about 9 percent. It should be noted that the curves for the muck come together at about 4 minutes, and the curve for test 1 is practically horizontal from that point on, indicating the probability of some sticking in the machine. Also, although the muck tests were continued for as long a time as the clay tests, they were plotted only to about 150 minutes because they are practically horizontal beyond that point. It is apparent from these tests that the final voids ratio is not appreciably dependent on the initial moisture content.

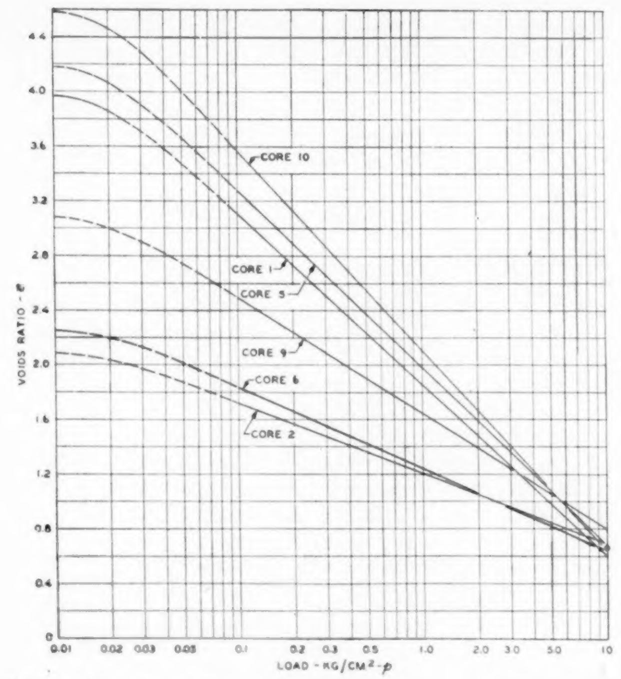

FIGURE 11.—LOAD-COMPRESSION CURVES OF UNDISTURBED CORE SAMPLES.

9. At any stage of loading, equilibrium between load and pores, as disclosed by the load-compression curve, exists throughout the depth of any muck layer.

10. The average time-compression curve that is used discloses the relation between duration of load increment and percentage of compression caused by any load increment within the load range of 0.01 ton per square foot and that required to compress the soil to its plastic limit.

A study of the time-compression data for different load increments discloses that no two of the several curves determined for each test are identical. However, variations between the curves are independent of load increment and no apparent relation exists. Generally the average of the several sets of data furnishes a curve within 10 percent of any individual curve at all time ordinates. Consequently, it seems practical to make use of this average curve.

FORM OF TYPICAL LOAD-COMPRESSION CURVES DETERMINED

Comprehensive study of all the compression test data furnished by the Bureau's investigations discloses that, when plotted to a semilogarithmic scale, the load-compression relation generally results in a curve that becomes a straight line for loads greater than 0.1 kilogram per square centimeter. Figure 11 shows the compression test data for the soils used in this analysis.

For values of p greater than 0.1 kilogram per square centimeter the data results in straight lines of the form

$$e=B-Z\log p \qquad (4)$$

in which B is the intercept of the straight-line portion of the curve at the 1.0 kilogram per square centimeter ordinate, and $-Z$ is the slope of the semilog plot of the curve, or range in value e for unit change of log p. Z may be termed the "compression index" of the soil.

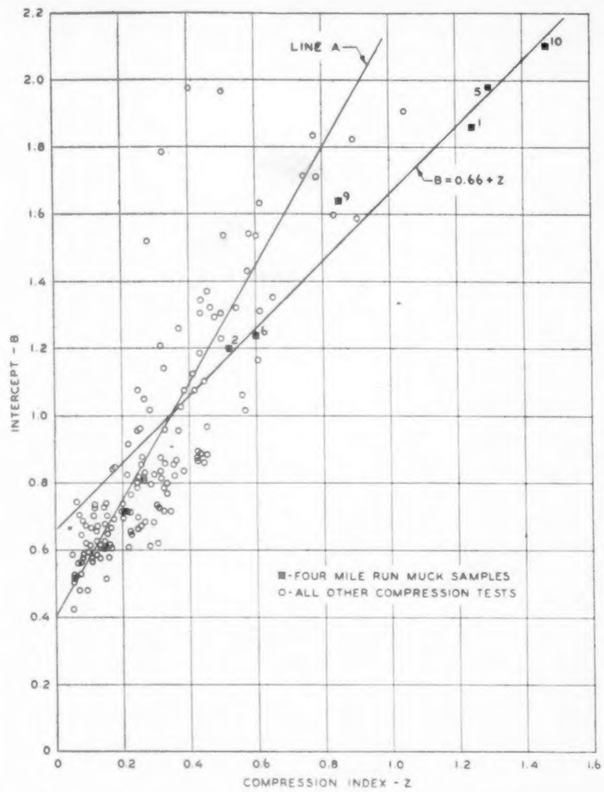

FIGURE 12.—RELATION BETWEEN COMPRESSION INDEX AND INTERCEPT.

The relation between the slopes and intercepts for the mucks used in this analysis are plotted in figure 12, and are found to conform closely to the formula

$$B=0.66+Z \qquad (5)$$

The relations between the slopes and intercepts of a great number of load-compression curves are also plotted in figure 12, and are very roughly represented by line A.

Figure 13 shows a very general relation between the compression index Z and the liquid limit of the mucks used in this analysis. This fact makes possible a short and approximate method of estimating settlements which will be discussed later.

For pressures less than about 0.1 kilogram per square centimeter the load-compression relation may vary from a horizontal line to a curve of form depending upon such factors as:

1. The degree of consolidation of the sample at the beginning of the test.
2. The degree to which the surfaces of the sample were disturbed during its transfer from the core container to the testing apparatus.
3. Experimental errors caused by friction or by characteristics of the testing apparatus.

Were the effect of these and possibly other influences removed, the straight-line portion of the semilog plot of the curve for loads greater than 0.1 kilogram per square centimeter could be extended for loads less than 0.1 kilogram per square centimeter until a maximum voids ratio equal to that of the soil immediately after

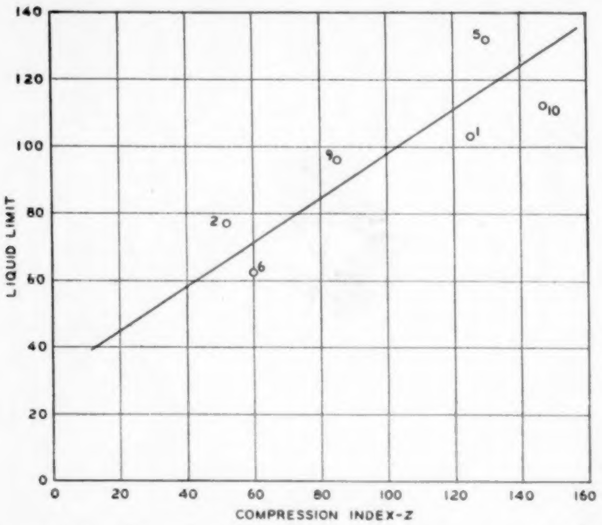

FIGURE 13.—RELATION BETWEEN LIQUID LIMIT AND COMPRESSION INDEX.

settling out of suspension is reached. Flocculation tests[9] performed on similar materials indicate that for a soil similar to that of core 5, the maximum voids ratio would be about 7.0.

This high voids ratio, however, exists but temporarily. A better starting point is the voids ratio that the top of a sediment will ultimately attain due entirely to the weight of the soil particles in a layer of single-particle thickness and not influenced by the load resulting from an accumulation of particles above this increment.

The exact value of this voids ratio would be difficult to determine in all instances. Therefore the compression curves plotted for all the tests performed in the Bureau's subgrade investigations were studied to determine some general form of curve that could be used for practical purposes to express, approximately, the relation of voids ratio to load for loads of less than 0.1 ton per square foot.

It was found that such a curve, extending to a voids ratio high enough to satisfy the requirements of practical application, has the form of the broken lines in figure 11. This curve is expressed by the following empirical formula, the numerical constants of which were evaluated by the method of least squares.

$$e=B+Z(1.69-1.07x^2+0.38x^3) \quad (6)$$

in which

e=voids ratio for loads between 0.01 and 0.10 kilogram per square centimeter.

B=intercept of the straight-line portion of the semilog plot of the curve at the 1.0 kilogram per square centimeter load ordinate.

$-Z$=slope of the semilog plot of the straight-line portion of the curve, or range in value e for unit change in log p.

$x=2+\log p$.

Using this formula the voids ratios corresponding to various loads between 0.01 kilogram per square centimeter and 0.10 kilogram per square centimeter for the mucks in this analysis are as shown in table 6.

The broken-line curves in figure 11, for loads smaller than 0.1 kilogram per square centimeter, and the solid-line curves for loads greater than 0.1 kilogram per square centimeter, are assumed, therefore, to express the relation of voids ratio to load between small or zero loads and loads at the plastic limits. There is a general tendency for all of the curves to intersect in a common point corresponding to a voids ratio of about 0.66 on the ordinate for 10 kilograms per square centimeter.

[9] For procedure see Subgrade Soil Testing Methods, by C. A. Hogentogler and E. A. Willis, proceedings of the A. S. T. M., 1934.

RELATIVE RATES OF CONSOLIDATION OF SOIL STRATUM AND SOIL SAMPLE COMPARED

The time-compression curves used in this analysis are shown as semilogarithmic graphs in figure 14. These curves are the averages of the individual time-compression curves for different load increments after certain ones had been eliminated because of obvious error.

Before using the curves in figure 14, in estimating the speed of consolidation of soil strata in place there must be determined the relative periods of load application at which equal percentages of compression occur in the soil strata and in their representative laboratory samples.

TABLE 6.—*Characteristics of compression curves for undisturbed muck samples*

Material from core no.	B	Z	Voids ratio, e, for the following loads					
			0.010 Kg per sq. cm	0.015 Kg per sq. cm	0.02 Kg per sq. cm	0.04 Kg per sq. cm	0.07 Kg per sq. cm	0.10 Kg per sq. cm
1	1.86	1.25	3.97	3.93	3.86	3.59	3.30	3.11
2	1.20	.52	2.08	2.06	2.03	1.92	1.80	1.72
5	1.98	1.30	4.18	4.14	4.06	3.78	3.48	3.28
6	1.24	.60	2.25	2.23	2.20	2.07	1.93	1.84
9	1.64	.85	3.08	3.05	3.00	2.82	2.62	2.49
10	2.10	1.47	4.58	4.54	4.46	4.14	3.80	3.57
Values of $(1.69-1.07x^2+0.38x^3)$ for these loads			1.69	1.66	1.60	1.38	1.15	1.00

The determination is made on the basis of the time required for a unit volume of water to travel a unit distance through the soil pores and for the conditions (see fig. 15) met in this report. The method is as follows:

Case 1.—Water free to flow from both top and bottom of the soil stratum.

Let e_1=voids ratio of both soil sample and soil stratum before compression, under pressure of p_1.

e_2=voids ratio of soil sample and soil stratum after compression by pressure p_2.

d_1=thickness of the soil sample at e_1. (See formula (1).)

D_1=thickness of the soil layer at e_1.

t=average time required for a unit volume of water to travel a unit distance through the soil during the change from e_1 to e_2.

t_d=time required for the soil sample to consolidate from e_1 to e_2.

t_D=time required for the soil layer to consolidate from e_1 to e_2.

The total units of water volume to be squeezed out of the laboratory sample per unit of cross-sectional area equals $\frac{e_1-e_2}{1+e_1}d_1$, one half of which flows out of each face. The average distance traveled by each unit of water equals $\frac{d_1}{4}$.

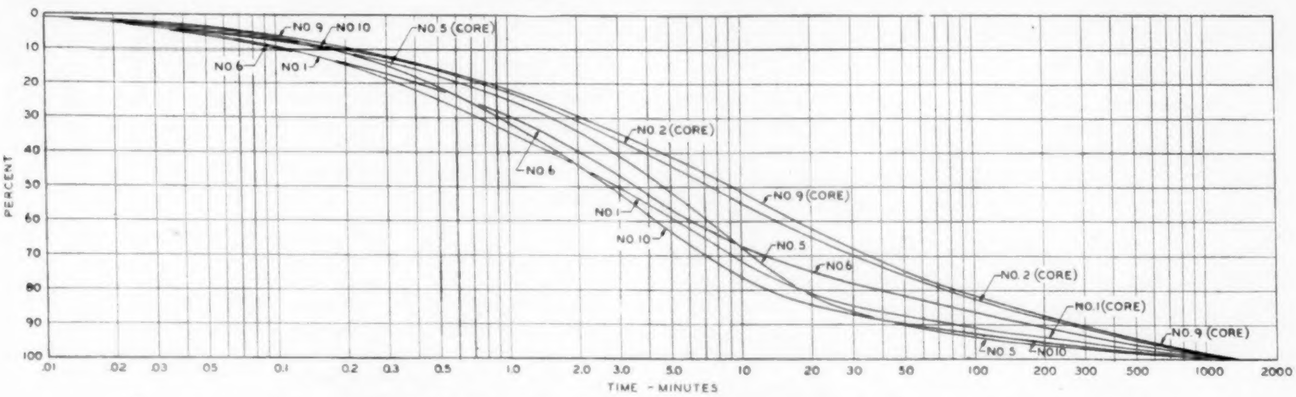

FIGURE 14.—AVERAGE TIME-COMPRESSION CURVES OF UNDISTURBED CORES FROM HYDRAULIC FILL.

Therefore the time required for the laboratory sample to consolidate from a voids ratio of e_1 to e_2 is

$$t_d=\frac{e_1-e_2}{1+e_1}\times\frac{d_1}{2}\times\frac{d_1}{4}t$$

In the same manner the time required for the soil stratum of thickness D_1 to consolidate an equal amount is

$$t_D=\frac{e_1-e_2}{1+e_1}\times\frac{D_1}{2}\times\frac{D_1}{4}\times t$$

Then

$$\frac{t_d}{t_D}=\frac{d_1^2}{D_1^2}\text{------------------} (7)$$

Case 2.—Water free to flow out of but one face, either top or bottom of the soil stratum.

In this case all of the water or $\frac{e_1-e_2}{1+e_1}\times D_1$ units per unit area of soil face must escape in one direction and the average distance it travels is $\frac{D_1}{2}$. Then the time of consolidation of the soil stratum becomes

$$t_D=\frac{e_1-e_2}{1+e_1}\times D_1\times\frac{D_1}{2}\times t$$

and

$$\frac{t_d}{t_D}=\frac{d_1^2}{4D_1^2}\text{------------------} (8)$$

STATE OF CONSOLIDATION OF MUCK SAMPLES PRIOR TO FILL CONSTRUCTION IMPORTANT

This analysis involves four distinct computations, which are: (*a*) State of consolidation of the muck deposit prior to the fill construction; (*b*) elevation of the top of the muck layer immediately after completion of the fill; (*c*) amount and rate of subsequent consolidation produced by the fill material; and (*d*) the moisture content of the muck at the locations from which cores were obtained.

The layer of muck at the location of boring S–6 from which core 5 was taken, had a bottom elevation of −15.9 feet when the borings were made and a top elevation of −3.0 before construction. Since it is not likely that the elevation of the firm sand under the muck changed appreciably during the construction of the fill it seems reasonable to assume that the thickness of the muck layer was 12.9 feet prior to construction.

The consolidation of the muck layer caused by the weight of the soil solids alone is estimated as follows:

At any voids ratio the weight of the solid material in a volume of the muck below water level is given by

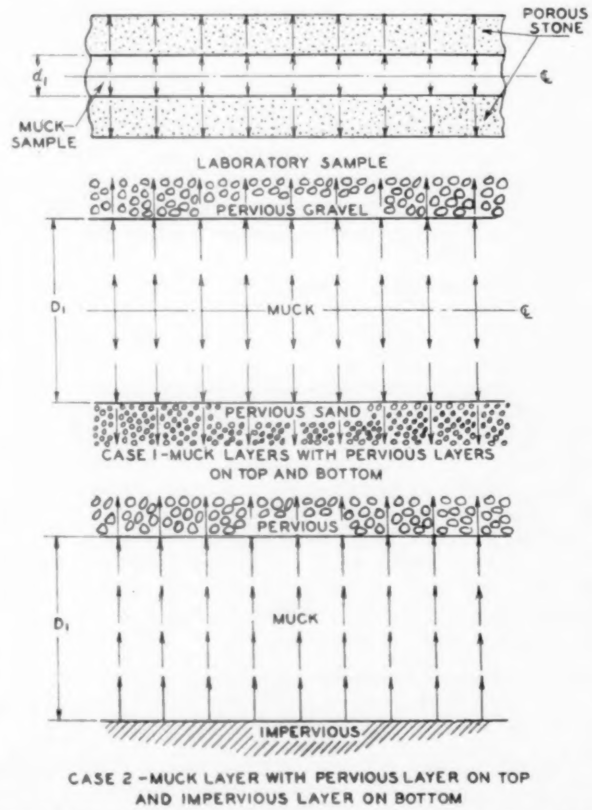

FIGURE 15.—MEANS BY WHICH WATER ESCAPES FROM MUCK IN LABORATORY SAMPLE AND IN FILL.

the formula

$$IW_s=V\frac{1}{1+e}(G_s-G_w)\text{------------------------} (9)$$

in which

IW_s = the weight of the immersed solid material.
V = the volume.
e = the voids ratio.
G_s = the specific gravity of the soil, in this case 2.6.
G_w = the specific gravity of water, in this case 1.

At elevation −3.0 under zero load the voids ratio for core 5, from the load-compression relation, table 6, is 4.18 and at a depth of 1 foot (30.48 centimeters) the pressure would be

$30.48\times\frac{1}{1+4.18}\times(2.6-1)\times\frac{1}{1,000}=0.0094$ kilograms per square centimeter.

This pressure, according to assumptions used in developing a standard form for the load-compression curve (see complete work sheet, fig. 16) cannot further consolidate the layer and consequently the voids ratio at elevation -4.0 is 4.18, the same as at elevation -3.0.

The combined weight of the first 2 feet of muck is $0.0094+0.0094$ or 0.0188 kilograms per square centimeter, which is the pressure acting at elevation -5.0. Proper substitution in formula (6), or reading directly on the corresponding curve, figure 16, shows that this load can consolidate the sample to a voids ratio of 4.08.

Continuation of this process provided the data of table 7 and figure 16D, which shows the pressures acting at all locations throughout the thickness of the muck deposit before the fill construction.

LENGTH OF TIME REQUIRED FOR COMPLETE FILL SETTLEMENT DETERMINED

The computations were based on the assumption that the muck bed was completely consolidated by its own weight. According to formula (7) the greater the thickness of the layer the longer will be the time required for complete consolidation. To test the validity of this assumption, the greatest possible length of time required for complete settlement must be known. For computing this, the maximum possible thickness that the soil deposit could ever have had must be used. This is derived as follows:

The average voids ratio of the deposit as determined by averaging the voids ratios shown in table 7 is 3.60. It may then be assumed that the voids ratio of the muck layer during the course of natural consolidation under its own weight changed from 4.18 to an average of 3.60. This means that every increment of thickness in the layer would have retained a voids ratio of 4.18 had it not been required to carry the load imposed by additional deposits. It may be assumed that the maximum thickness of the deposit never exceeded that represented by a voids ratio of 4.18.

TABLE 7.—*Degree of consolidation of material prior to construction at point where core 5 was taken*

Elevation (feet)	Thickness	Total muck load (p)	Voids ratio	Load increment per foot depth
	Feet	*Kg per sq. cm*		*Kg per sq. cm*
−3	0	0	4.18	0.0094
−4	1	0.0094	4.18	.0094
−5	2	.0188	4.08	.0096
−6	3	.0284	3.94	.0099
−7	4	.0383	3.80	.0102
−8	5	.0485	3.68	.0104
−9	6	.0589	3.58	.0106
−10	7	.0695	3.49	.0109
−11	8	.0804	3.40	.0111
−12	9	.0915	3.33	.0113
−13	10	.1028	3.27	.0114
−14	11	.1142	3.21	.0116
−15	12	.1258	3.15	.0117
−15.9	12.9	.1363	3.11	(1)
Average			3.60	

1 $(0.9\times0.0117=0.0105.)$

If the muck layer, with an original voids ratio of 4.18, were compressed to a voids ratio of 3.60 and a thickness of 12.9 feet, then its original thickness may be computed from the formula

$$\frac{1+e_1}{1+e_2}\times D_2=D_1 \qquad (10)$$

in which e_1 is the original voids ratio, in this case 4.18, corresponding to a thickness of D_1, and e_2 is the voids ratio of the compressed material, in this case 3.60, corresponding to a thickness, D_2, of 12.9 feet.

Then

$$D_1=\frac{1+4.18}{1+3.60}\times12.9=14.5 \text{ feet.}$$

The manner in which the formation of the muck deposit took place is a matter of conjecture. To estimate the time of ultimate settlement of the muck deposit, it may be assumed that the layer was originally 14.5 feet thick with a uniform voids ratio of 4.18. According to the test data the thickness of the laboratory sample at this voids ratio would have been 0.51 inch and equilibrium under its own weight was reached in 1,200 minutes. The time required for the consolidation of the muck deposit is computed from formula (7) as follows:

$$t_D=\frac{t_d\times D_1^2}{d_1^2}=\frac{1,200\times(12\times14.5)^2}{0.51^2}=140,000,000 \text{ minutes}$$

or about 270 years.

Geologists have estimated the age of the muck layer at Four Mile Run as several thousand years. According to what precedes, it would take about 270 years for the completion of consolidation caused by its own weight.[10] Consequently, the assumption that 100 percent consolidation has occurred within the muck layer at the location of core 5, prior to fill construction, is well founded.

Similar computations were made for boring S–9, located 264 feet to the left of the centerline, station 183+00, where core 6 was obtained. The computed time of complete settlement at this location was 2,230 years, the greatest for any location at which borings were made. This is considered as within the estimated age of the muck. Therefore, the assumption that complete consolidation caused by its own weight had occurred throughout the muck layer prior to fill construction is consistent.

TYPICAL METHOD OF ANALYSIS FOR SECTION UNDER FILL LOAD

According to table 7 the average voids ratio of the muck layer at boring S–6 is 3.60, which corresponds to a load of 0.057 kilogram per square centimeter acting until consolidation under this pressure is complete.

The results of borings made 6 months after fill construction are shown in figure 16C. According to this figure the muck at this time supported a gravel layer 9.0 feet thick and had a top elevation of -4.9 feet as compared with -3.0 feet before the improvement was begun.

Assuming that the gravel layer does not consolidate, it will have a constant thickness of 9 feet acting continuously after fill construction. Consequently, the difference between -4.9 and -3.0 must be accounted for by a change in thickness of the muck layer.

The muck will, of course, consolidate under load.

10 This value is not to be confused with the time of ultimate consolidation under the fill of 243 years subsequently determined. The value of 270 years represents the computed time for complete consolidation of the muck caused by its own weight acting within the layer itself. It is based on a layer 14.5 feet thick. The value of 243 years represents the time required for the muck layer to reach complete consolidation due to the superimposed gravel fill, and is based on a layer only 12.2 feet thick.

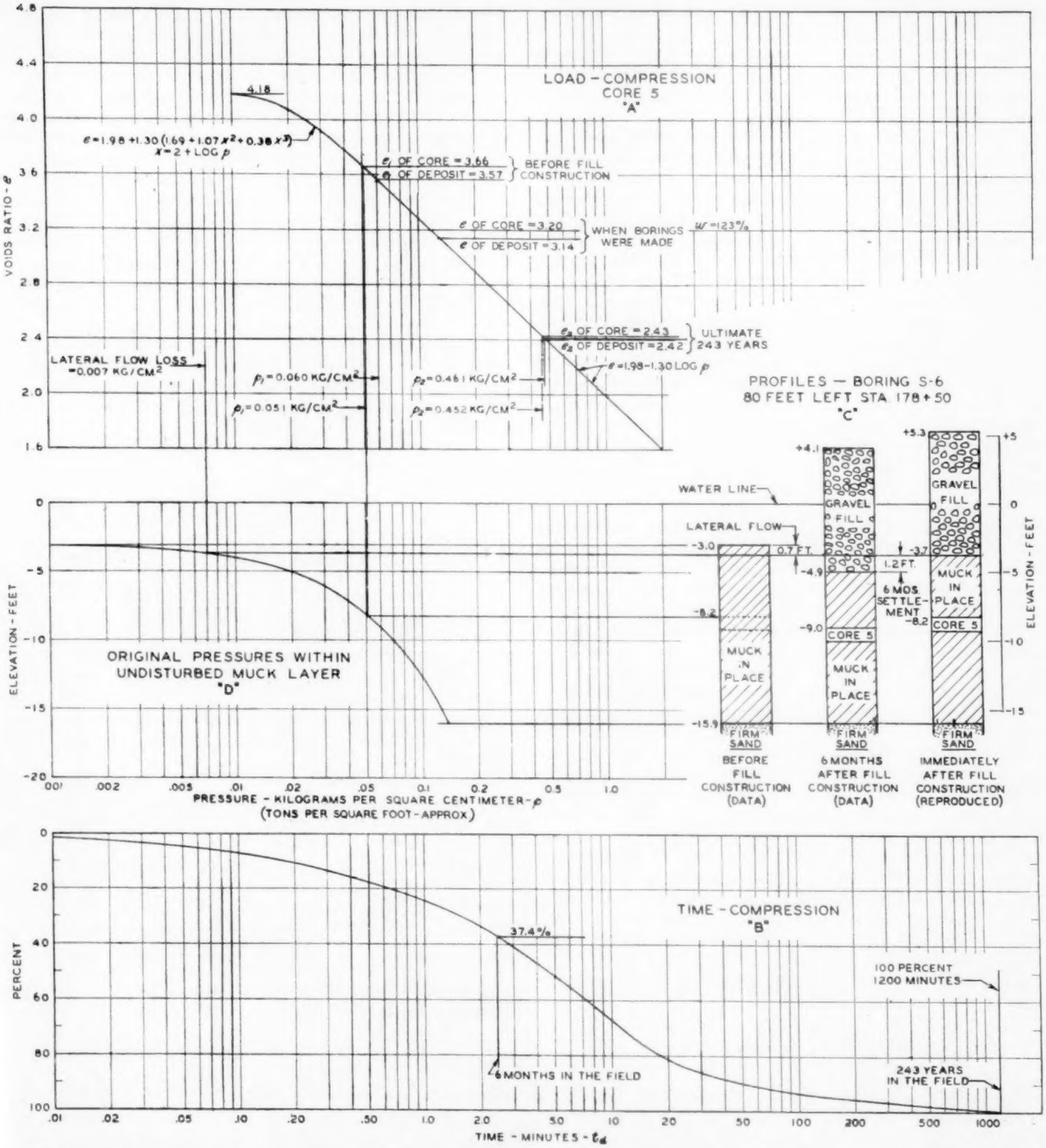

Figure 16.—Graphical Analysis of the Compression Test Data of Undisturbed Material from Core No. 5.

Furthermore, there is a possibility that during the pumping operations a portion of the muck layer was displaced laterally. No measurements were made to determine the extent of lateral flow during construction. The thickness of the muck layer displaced in this manner, therefore, must be determined by a series of approximations before the analysis of settlement can be completed.

Had no lateral flow occurred, the gravel fill when first constructed would have rested on top of the muck layer shown in figure 16C, left. The effect of the fill acting for 6 months would be to reduce the thickness of the muck layer by an amount indicated by the load-compression and time-compression curves.

The thickness of the muck layer disclosed by the boring subtracted from the computed reduced thickness

gives an estimate of the thickness of the material displaced by lateral flow.

Because of this loss of material both the thickness of the muck layer and the load acting on it immediately after construction are less than those used in the first computation. Consequently, a second approximation is made using a revised thickness and load. This process is repeated until the computed thickness practically equals that determined by the borings.

The results of such computations, shown in figure 16, were obtained in the following manner. The load imposed by the gravel fill, 6 feet above water at 110 pounds per cubic foot and 3 feet below water at 68.5 pounds per cubic foot was computed to be 0.422 kilogram per square centimeter. The average voids ratio of 3.60 referred to above equals that produced by a load of 0.057 kilogram per square centimeter compressing the sample for a long time.

This load of 0.057 kilogram per square centimeter is not the average load acting throughout the depth of the deposit. It is the load acting on top of a layer of zero weight that will produce a voids ratio equal to the average voids ratio produced by the variable load distributed through the deposit.

The total effective load acting on top of the layer is $0.057+0.422=0.479$ kilogram per square centimeter. This will ultimately consolidate the sample to an average e_2 of 2.40 (equation (4) or fig. 16). The difference in voids ratio produced by the additional load equals $3.60-2.40$ or 1.20.

The percentage of consolidation which will occur in 6 months may be obtained from formula (7) and the time-compression curve. The thickness, D_1, of the muck layer was 12.9 feet. The thickness of the laboratory sample, d_1, corresponding to the average voids ratio of 3.60 is 0.452 inch. The time of settlement of the muck layer in place, t_D, is 6 months. Substitution of these values in formula (7) indicates that the time in the laboratory, t_d, in which the same percentage of compression will be accomplished, is 2.24 minutes. This according to the time-compression curve, figure 16B, corresponds to 35.7 percent. Consequently, the amount of additional compression accomplished in 6 months is represented by a change in voids ratio of 35.7 percent of 1.20 or 0.43. The reduced voids ratio of the muck layer is therefore the initial voids ratio 3.60 minus the reduction 0.43, or 3.17.

This is indicative of a thickness of muck layer 6 months after fill construction of

$$12.9\times\frac{1+3.17}{1+3.60}=11.7 \text{ feet}$$

The boring (fig. 16C, middle) shows that the muck layer was actually 11.0 feet thick at this time. It is apparent, therefore, that approximately 0.7 foot of muck had been laterally displaced. At what point or points within the muck layer this amount was lost is not known. However, it is obvious that the least stable portion of the muck would be displaced. The top of the layer has the highest voids ratio, according to table 7, and therefore the least density and least resistance to lateral flow. In subsequent computations, it is assumed that 0.7 foot of muck was removed from the top of the layer.

The second approximation is therefore made on the assumption that the muck layer is only 12.2 feet thick instead of 12.9 feet thick. This has the effect of increasing the amount of gravel under water to 3.7 feet and decreasing the amount of gravel above water to 5.3 feet. The load produced by the gravel in this position is 0.408 kilogram per square centimeter.

A further effect of the removal of 0.7 foot of muck is a change in the average voids ratio of the muck layer, as the material removed according to preceding assumptions had a voids ratio of 4.18 against the average of 3.60. This results in a lower average voids ratio, equaling 3.57 in this case, corresponding to a compression load of 0.060 kilogram per square centimeter. The weight of the displaced muck equals 0.7×0.0094 or 0.007 kilogram per square centimeter. The total load is thus $0.408 + 0.060 - 0.007$ or 0.461 kilogram per square centimeter. According to the load-compression curve, figure 16A, this load will ultimately produce a voids ratio of 2.42.

Continuing the computations as in the first approximation, it is found that the reduced thickness of muck layer, 6 months after fill construction, becomes 11.0 feet, which is the thickness as determined from the borings.

Consequently, the revised profile showing conditions immediately after fill construction (fig. 16C, right) is constructed on the basis of a muck layer 12.2 feet thick.

SETTLEMENT ANALYSIS COMPLETED AND MOISTURE CONTENT OF SAMPLE ESTIMATED

In the same manner that the thickness of 11.0 feet was computed for a period of settlement of 6 months under a load of 0.461 kilogram per square centimeter, thicknesses may be computed for any desired period up to the time of ultimate settlement. By proper substitutions in formula (7) the period of time required for ultimate settlement, $e_2=2.42$, may also be determined. Results of such computations are shown in table 8.

The moisture content of core 5 was determined from a sample taken from the top of the core shown in figure 16C, middle, at an elevation of −9.0, or 6.9 feet above the sand layer. The core was taken 6 months after fill construction. The muck layer, 12.2 feet thick immediately after fill construction, had consolidated to a thickness of 11.0 feet. Consequently, the position of the top of sample 5 immediately after fill construction was approximately $\frac{6.9\times 12.2}{11.0}$ or 7.7 feet above the sand layer, which corresponds to an elevation −8.2 as shown in figure 16C, right.

TABLE 8.—*Computed settlement for different periods of time*

Period of settlement (months)	Average voids ratio	Consolidation	Computed thickness	Computed settlement
		Percent	*Feet*	*Feet*
0	3.57	0	12.2	0
1	3.38	16.1	11.7	0.5
6	3.14	37.4	11.0	1.2
12	2.98	51.2	10.6	1.6
20	2.84	62.8	10.3	1.9
36	2.69	76.1	9.9	2.3
Maximum [1]	2.42	100.0	9.1	3.1

[1] Occurs in 243 years' time.

From table 7, the voids ratio at elevation −8.2 is found by interpolation to be 3.66, which corresponds to a load of 0.051 kilogram per square centimeter on the load-compression curve, figure 16A. This, minus the load of 0.007 kilogram per square centimeter lost by lateral flow and added to the fill load of 0.408 kilogram

per square centimeter, makes a total load of 0.452 kilogram per square centimeter acting on the sample. Ultimately this will consolidate the sample to a voids ratio of 2.43, causing a decrease in voids ratio equal to 3.66−2.43, or 1.23.

The percentage of consolidation that occurs in 6 months in this sample is 37.4 percent or the same as that occurring in the entire layer as shown in table 8. Therefore, the voids ratio of the sample after 6 months equals $3.66-(1.23\times0.374)=3.20$. This corresponds to a moisture content of $\frac{3.20}{2.6}\times100$, or 123 percent, as compared with the moisture content of 125 percent as actually determined in the sample.

The computed moisture contents shown in table 2 and the computed settlement shown in table 3 and figure 7 were determined in the manner described above.

METHODS OF COMPUTING FILL SETTLEMENT DISCUSSED

The method of computing the settlements in this report was adopted after much preliminary work in which other somewhat similar methods were tried and rejected after analyzing their suitability and reliability.

Most of the work of estimating probable settlement of loaded soil layers rightfully belongs in the field of foundation engineering. Problems in this field generally differ from that studied at Four Mile Run, since the loads are transmitted from building to soil by footings, rafts, piling, etc. There is no uniform loading, such as is produced by a fill, and the compressible layers are usually under appreciable topsoil overburden. This makes the determination of the pressures quite a difficult problem and lessens the importance of variations in voids ratios in the layer. The voids ratio of a core taken within the layer is generally assumed as the voids ratio of the layer. Additional data on foundation settlement estimation from compression tests have been published.[11]

A simpler method of computing settlements is to use the natural moisture content and corresponding voids ratio of the undisturbed sample as the initial voids ratio, e_1, and determine the ultimate voids ratio, e_2, to which the layer would compress under load, directly from the load-compression curve. Having the total settlement, one can determine the settlement occurring after any given time interval.

This method gives fairly accurate results for most of the locations discussed in this report. However, research along more accurate lines had to be performed to determine the validity of various methods. For study of conditions at Four Mile Run the more accurate method seemed desirable. The muck bed at that location was sampled after it had undergone some 6 months of compression under the fill load instead of prior to construction, as would be the case in a general investigation of settlement. This alone indicated a need for a special method of computation.

The wide range of voids ratios existing in the layer from top to bottom was a further indication of the need for the more accurate procedure. According to table 7 the theoretical voids ratio of a sample from elevation −5.0 would have been about 4.08 prior to fill construction. A sample from elevation −15.0 would be expected to have a voids ratio of 3.15. In the case of a deeper layer the voids ratio of the muck at elevation −40.0 would be about 2.40. The actual core sample in this case was taken from the muck at elevation −9.0 after 6 months compression under load. Calculations based on the moisture-content determination indicate that this material was at elevation −8.2 prior to fill construction. Table 7 shows that sample 5 would have had a voids ratio of about 3.66 at that time, corresponding to a natural moisture content of 141 percent.

This value happens to be quite close to the average voids ratio of 3.60 determined for the layer. Therefore, in this particular case there is little difference in the settlement determined by either method. However, the use of test results from core 5 material in the computations for all points used where settlement comparisons were made would have necessarily yielded much more inaccurate results for the thicker portions of the muck bed, since the average initial voids ratios decrease with an increase in thickness. The settlement estimated by this simple method decreases as the voids ratio of the sample decreases. The degree of error introduced by the method depends on the difference between the voids ratio of the sample and the average voids ratio of the deposit.

For example, the total settlement at the location of core 5 based on a sample taken at elevation −5.0 equals about 32 percent of the thickness; that based on a sample taken at elevation −15.0 is only 21 percent of the layer thickness. In this instance the settlement estimated by the simpler method (sample taken at elevation −8.2) becomes 27 percent as compared to the value determined by the adopted method, of about 25 percent. It is interesting to note that the computed settlement based directly on core 5 test data without a 6-months correction is 21 percent of the layer thickness, or that indicated by the material sampled at −15.0.

TABLE 9.—*Settlement computations by 1-foot increments for boring S–6 based on core no. 5*

Elevation (feet)	Thickness	Total muck load	Voids ratio	Average voids ratio for each foot	Total muck load (averaged)	Total load gravel and muck load	Resulting voids ratio	Resulting thickness of 1-foot layer
	Feet	*Kg per sq. cm*			*Kg per sq. cm*	*Kg per sq. cm*		*Feet*
−3	0	0	4.18			0.4225		
−4	1	0.0094	4.18	4.18	0.0047	.4272	2.47	0.67
−5	2	.0188	4.08	4.13	.0141	.4366	2.46	.67
−6	3	.0284	3.94	4.01	.0236	.4461	2.44	.69
−7	4	.0383	3.80	3.87	.03335	.45585	2.43	.70
−8	5	.0485	3.68	3.76	.0434	.4659	2.41	.72
−9	6	.0589	3.58	3.63	.0537	.4762	2.40	.73
−10	7	.0695	3.49	3.535	.0642	.4867	2.39	.75
−11	8	.0804	3.40	3.445	.07495	.49745	2.37	.76
−12	9	.0915	3.33	3.365	.08595	.50845	2.36	.77
−13	10	.1028	3.27	3.30	.09715	.51965	2.35	.78
−14	11	.1142	3.21	3.24	.1085	.5310	2.33	.79
−15	12	.1258	3.15	3.18	.1200	.5425	2.32	.79
−15.9	12.9	.1363	3.11	3.13	.13105	.55355	2.31	.72
Average or total			3.60				2.39	9.54

Table 9 presents evidence that the use of the average voids ratio, as determined in table 7, in computing settlements gives the same result as a summation of the compression of each 1-foot increment determined separately. In table 9 the average voids ratios for each 1-foot layer are given with the corresponding load increments and total loads. The resulting ultimate voids ratios are determined from the load-compression relation and the compressed thickness of each increment is obtained in the same manner as before. A value of

[11] Progress Report of Special Committee on Earth and Foundations, Proceedings American Society of Civil Engineers, May 1933.

total ultimate thickness of 9.54 feet is obtained as compared with the value of 9.53 feet obtained by the use of the average voids ratio of the entire layer. The average of the final voids ratios derived by this foot-by-foot determination is 2.39, while that determined from the average voids ratio of the entire layer is 2.40.

SETTLEMENTS MAY BE ESTIMATED BY AN APPROXIMATE METHOD

The relations established in the foregoing analysis suggest an approximate method for rapidly estimating settlements in similar deposits. It appears that a knowledge of the moisture content of the material in its natural state and of the type of fill to be constructed, combined with probe borings to disclose the depth of the muck layer, furnish sufficient information for a rough estimate.

This could be carried out in some such manner as follows:

1. Make sufficient probe borings to determine the thickness of the muck layer.
2. Obtain several samples of the muck for moisture content determination and routine tests, if desired. These samples can be taken during probe boring by driving a pipe smaller than the casing into the soil just below the elevation penetrated by the casing. Samples should be obtained near the middle of the layer.
3. Determine the moisture content of each sample. Use the highest value and assuming that this value represents the moisture content at the liquid limit, determine the compression index, Z, from figure 13. An average value might be used but the highest value is suggested as being on the safe side. If the moisture content of one of several samples varies considerably from the others, it should be discarded.
4. From formula (5) determine value of intercept B. From figure 11 select the load-compression curve nearest the determined values of B and Z. If no curve fits the values closely, or if otherwise desired, the load-compression curve may be plotted since the data are sufficient to fix the position of the curve.
5. Determine the voids ratio corresponding to the natural moisture content, using a specific gravity of 2.6. Determine the equivalent load from the selected load-compression curve. Add the amount of superimposed load to be used, thus obtaining the total load. Read off the ultimate voids ratio from the chosen curve. Compute the maximum settlement.
6. Using values of 0.4 inch for d_1, and known field thickness, D_1, in formula (7), determine the laboratory time interval corresponding to the selected field time intervals. Read the corresponding percentages off of figure 14. Any of the six time-compression curves may be used but it is suggested that all six be considered as a band, and for each time interval the bottom of the band be read since this will give higher percentages and thus any error will be on the side of safety. Determine the settlement for the selected time interval.

The following example will serve to illustrate this short approximate method.

Probe borings disclose that a muck layer about 15 feet thick exists at the site of a proposed fill. The muck rests on a firm sand and is entirely under water. The proposed fill will produce a pressure of 1.5 tons per square foot. The natural moisture content of the muck is found to vary from 100 to 120 percent. Using this latter value, the compression index, Z, of this soil is about 1.33 according to figure 13. From formula (5)

$$B=0.66+Z=0.66+1.33 \text{ or } 1.99.$$

According to figure 11 these values indicate a load-compression curve closely approaching that for core 5 material. Consequently, that curve can be used. Using a natural moisture content of 120 percent, which corresponds to an initial voids ratio e_1, of 3.12, the sample is found to be compressed by an equivalent load of about 0.13 ton per square foot, as read off the curve for core 5 material, figure 11. The total load is thus 1.50+0.13 or 1.63 tons per square foot. According to the curve being used, this load is capable of compressing the muck to a voids ratio, e_2, of about 1.70. The ultimate settlement for an initial thickness, D_1, of 15 feet is

$$\frac{e_1-e_2}{1+e_1}\times D_1=\frac{3.12-1.70}{4.12}\times 15=5.2 \text{ feet.}$$

Substituting in formula (7) for the laboratory time period corresponding to 6 months in the field

$$t_d=6\times 30\times 1{,}440\frac{(.4)^2}{(15\times 12)^2}=\text{about 1.3 minutes.}$$

According to figure 14 the maximum consolidation occurring in any of the Four Mile Run muck samples in the laboratory at 1.3 minutes was about 38 percent. Therefore the estimated settlement likely to occur in 6 months is 38 percent of 5.2 feet, or 2.0 feet. By the same procedure the settlement after 1 year is estimated at about 48 percent or 2.5 feet, and the settlement after 5 years is about 80 percent of the maximum or about 4.2 feet. Similarly the settlement after any desired time can be determined.

The foregoing is not recommended as an accurate method but will give a rough estimate of settlements likely to occur in muck beds similar in character and subjected to loading of the character discussed in this report. In any case, it is desirable to perform the laboratory tests indicated in table 1 and to compare the results with those given in this table, and plotted in figures 3A and 3B. If the soil in question shows a considerable variation from these results, this method should not be employed.

STATUS OF FEDERAL AID HIGHWAY PROJECTS

(1936 FUNDS)

AS OF JANUARY 31, 1936

STATE	APPORTIONMENT	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 2,604,320										$ 2,604,320
Arizona	1,781,347	$ 52,617	$ 36,918	0.8	$ 1,812,737	$ 1,497,238	100.4	$ 15,723	$ 8,138	0.1	239,053
Arkansas	2,142,723										2,142,723
California	4,756,959	52,448	30,264	1.5	3,279,733	1,888,869	44.5	886,816	505,147	22.2	2,332,679
Colorado	2,288,811	25,046	14,026	.1	1,761,938	986,685	78.0	403,797	226,126	10.1	1,061,974
Connecticut	791,253										791,253
Delaware	609,375							422,476	209,443	38.2	399,932
Florida	1,655,723				674,678	337,339	23.8	218,968	109,484	8.8	1,208,900
Georgia	3,168,221				704,612	346,081	48.4	1,551,665	775,832	90.1	2,046,308
Idaho	1,531,162	305,432	172,421	44.6	978,940	583,705	95.0	454,955	272,245	33.0	502,791
Illinois	5,160,696				2,965,201	1,482,601	59.2	2,985,054	1,492,527	51.1	2,185,568
Indiana	3,087,613	237,057	118,523	6.9	3,221,269	1,606,817	110.3	1,962,072	980,947	51.6	381,326
Iowa	3,231,718	224,362	105,460	57.5	4,043,360	1,905,165	173.8	1,745,899	820,350	74.6	400,743
Kansas	3,317,054	9,038	4,519	5.3	551,929	275,964	113.8	2,546,000	1,273,000	261.8	1,763,571
Kentucky	2,304,143	375,048	187,524	19.7	1,661,657	811,246	111.6	1,306,165	617,178	49.3	688,195
Louisiana	1,776,939				959,642	479,821	35.7	1,175,105	587,552	40.7	709,566
Maine	1,090,167	113,605	56,802	2.1	1,504,065	752,033	49.6	94,663	47,332	1.9	234,000
Maryland	1,025,870										1,025,870
Massachusetts	1,741,877							396,055	198,028	3.1	1,543,849
Michigan	3,837,292	815,500	407,750	25.2	3,315,218	1,654,884	130.4	1,574,450	787,225	66.5	987,433
Minnesota	3,423,306	334,238	143,116	12.2	2,869,146	1,202,522	166.3	1,265,895	472,116	135.5	1,605,552
Mississippi	2,136,524										2,196,524
Missouri	3,800,856	1,171,810	585,905	220.8	3,127,048	1,563,524	116.0	393,968	196,984	31.6	1,454,443
Montana	2,560,449	691,680	387,272	141.9	938,454	525,440	84.3	1,173,316	656,940	70.5	990,797
Nebraska	2,581,663	1,116,878	558,439	66.5	605,827	302,913	68.2	49,779	44,252	5.2	1,676,059
Nevada	1,595,501	121,853	105,488	55.4	883,442	764,796	95.8				725,217
New Hampshire	609,375				781,661	384,377	20.9	32,264	16,082	1.1	208,916
New Jersey	1,675,751	37,143	18,572	.8	1,424,439	663,995	25.7	1,951,204	975,602	13.9	17,582
New Mexico	1,999,299	67,052	40,781	10.9	1,437,037	880,140	120.1	711,116	432,501	38.9	645,877
New York	6,150,106				7,686,056	3,830,200	143.2	1,997,400	953,250	31.4	1,366,656
North Carolina	2,938,657				1,124,169	561,992	189.6	43,935	21,968	9.0	2,354,697
North Dakota	1,960,162										1,960,162
Ohio	4,565,435				671,412	335,607	7.9	2,776,000	1,387,000	38.4	2,842,828
Oklahoma	2,947,521				1,450,144	762,050	50.4	423,581	220,337	11.6	1,965,134
Oregon	2,044,633				2,181,807	1,316,422	86.5	918,951	560,560	13.0	167,651
Pennsylvania	5,348,062	18,018	9,009	.5	3,971,136	1,985,568	55.9	2,157,858	1,078,592	33.1	2,274,893
Rhode Island	609,375										609,375
South Carolina	1,692,896										1,692,896
South Dakota	2,036,775				516,850	283,389	35.2	67,886	37,222	9.9	1,716,164
Tennessee	2,638,159	43,106	21,553	1.8	874,764	437,382	41.9	439,452	219,726	15.1	1,959,498
Texas	7,777,504	335,741	167,700	19.9	5,741,358	2,849,788	323.0	1,591,479	795,304	84.5	3,964,712
Utah	1,410,752	267,727	193,299	17.0	642,609	453,289	50.0	617,410	379,769	35.4	384,395
Vermont	609,375	25,988	12,994	2.9	1,093,744	546,872	50.8	52,043	26,021	4.7	23,488
Virginia	2,278,475				338,258	169,129	8.5	934,449	467,224	37.2	1,642,122
Washington	1,949,957	637,523	335,800	18.8	1,342,526	705,649	50.6	458,822	241,600	1.4	666,908
West Virginia	1,356,793	87,161	43,581	3.0	490,373	245,186	29.1	211,653	105,826	9.8	962,200
Wisconsin	3,045,557	235,767	117,883	4.8	1,451,933	724,830	71.6	497,253	246,034	14.4	1,956,810
Wyoming	1,559,444	527,052	324,603	83.6	1,968,673	1,212,486	191.0				22,355
District of Columbia											
Hawaii	609,375							229,826	114,641	2.5	494,734
TOTALS	121,875,000	7,928,890	4,200,202	824.5	71,047,845	37,315,994	3,257.0	36,735,403	18,560,105	1,451.2	61,798,699

CURRENT STATUS OF UNITED STATES WORKS PROGRAM HIGHWAY PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF JANUARY 31,1936

STATE	APPORTIONMENT	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	
Alabama	$ 4,151,115				$ 1,296,115	$ 1,296,115	37.1	$ 2,147,050	$ 2,147,050	70.8	$ 707,950
Arizona	2,569,841	$ 84,628	$ 84,628	11.0	1,184,660	1,114,773	67.6	530,314	411,917	28.3	958,522
Arkansas	3,352,061				1,191,239	1,189,621	86.5	847,757	846,294	79.5	1,316,145
California	7,747,928				2,260,950	2,136,474	98.9	2,953,704	2,859,512	95.0	2,751,942
Colorado	3,395,263	20,152	20,152	.7	1,017,393	1,017,002	66.1	309,829	309,829	13.4	2,048,280
Connecticut	1,418,709										1,418,709
Delaware	900,310				181,520	181,520	19.2	314,050	314,050	34.2	404,740
Florida	2,597,144				885,073	867,200	30.9	550,249	550,249	12.1	1,179,695
Georgia	4,988,967				47,181	47,181	.7	426,627	426,627	28.4	4,515,158
Idaho	2,222,747				974,913	953,356	84.0	236,081	200,935	15.3	1,068,456
Illinois	8,694,009				2,525,295	2,525,295	109.3	3,200,524	3,200,524	226.2	2,968,190
Indiana	4,941,255				714,325	714,325	35.2	3,031,947	3,031,947	182.9	1,194,983
Iowa	4,991,664				755,852	721,415	55.7	1,243,423	1,181,899	197.5	3,088,350
Kansas	4,994,975				1,580,386	1,580,386	172.0	947,105	946,105	98.9	2,468,484
Kentucky	3,726,271				699,705	699,705	100.8	1,766,866	1,748,811	201.8	1,277,755
Louisiana	2,890,429							817,186	544,923	36.0	2,345,506
Maine	1,676,799				704,322	704,322	33.4	621,785	621,237	26.3	351,241
Maryland	1,750,738							254,866	254,866	10.5	1,495,872
Massachusetts	3,262,885							115,246	115,246	1.1	3,147,639
Michigan	6,301,414	452,100	452,100	22.2	4,477,221	4,477,221	216.7	1,104,100	1,044,070	41.1	328,023
Minnesota	5,277,145	55,973	55,973	37.5	911,041	837,660	66.4	1,678,837	1,506,273	419.7	2,877,239
Mississippi	3,457,552				1,034,282	1,031,897	59.4	1,129,372	1,129,372	70.0	1,296,283
Missouri	6,012,652	28,173	28,173	14.3	1,758,680	1,758,680	386.4	1,622,254	1,622,254	302.6	2,603,545
Montana	3,676,416				1,959,107	1,959,107	110.8	1,027,895	1,027,895	63.0	689,414
Nebraska	3,870,739				993,968	989,167	102.1	1,263,046	1,230,595	120.3	1,650,977
Nevada	2,243,074	13,570	13,570		1,326,226	1,314,226	54.2	95,609	83,609	1.0	831,668
New Hampshire	945,225				257,318	255,212	7.3	163,250	104,458	4.6	585,555
New Jersey	3,129,805				507,749	507,749	8.5	1,183,286	1,183,286	4.4	1,438,770
New Mexico	2,871,397				986,014	986,014	89.9	554,130	554,130	26.1	1,331,253
New York	11,046,377				4,132,138	4,076,148	32.9	2,094,470	1,908,559	31.2	5,061,670
North Carolina	4,720,173				1,172,225	1,172,225	109.3	527,026	524,055	32.6	3,023,894
North Dakota	2,867,245	140,150	140,150	20.8	328,520	328,520	48.4	98,905	98,905	17.8	2,299,670
Ohio	7,670,815				1,201,932	1,201,932	13.6	1,330,300	1,319,300	11.7	5,149,583
Oklahoma	4,580,670				466,267	466,267	37.7	1,869,249	1,869,249	184.9	2,245,154
Oregon	3,038,642				1,478,728	1,468,728	44.0	752,299	735,034	84.9	834,880
Pennsylvania	9,347,797				149,696	149,696	.1	787,076	787,076	34.0	8,411,025
Rhode Island	989,208				113,368	113,368	4.0	546,895	543,228	11.0	332,612
South Carolina	2,702,012				296,459	296,459	25.2	468,873	468,713	50.3	1,936,840
South Dakota	2,976,454	79,687	79,687	30.1	590,313	590,313	152.3	360,270	360,270	68.4	1,946,184
Tennessee	4,192,460				505,406	505,406	22.6	611,494	611,494	26.1	3,075,560
Texas	11,989,350	8,067	7,769	4.9	4,384,793	4,220,319	392.9	2,831,504	2,790,062	233.5	4,971,201
Utah	2,057,154	27,439	27,439		587,011	566,399	51.6	348,285	318,611	29.6	1,154,705
Vermont	924,306				240,481	235,181	7.2	371,792	347,077	9.3	342,048
Virginia	3,652,667	39,453	39,453	28.9	886,865	886,865	475.6	572,218	572,218	209.7	2,154,132
Washington	3,026,161				1,645,583	1,586,948	81.9	766,611	628,279	51.4	810,934
West Virginia	2,231,412				574,968	574,968	30.3	311,060	311,060	9.1	1,345,383
Wisconsin	4,823,884	60,230	50,000	1.3	1,422,212	1,219,083	61.3	2,094,528	1,953,521	168.2	1,601,280
Wyoming	2,219,155				1,010,633	1,010,628	52.6	672,782	672,774	60.8	535,753
District of Columbia	949,496	238,253	238,253	1.9	525,643	501,613	4.1				209,631
Hawaii	926,033				332,074	318,456	4.3	154,448	151,751	1.6	455,826
TOTALS	195,000,000	1,247,875	1,237,347	173.6	52,275,850	51,355,145	3,751.6	47,706,473	46,169,199	3,737.1	96,238,309

U. S. GOVERNMENT PRINTING OFFICE: 193

CURRENT STATUS OF UNITED STATES WORKS PROGRAM GRADE CROSSING PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF JANUARY 31, 1936

STATE	APPORTIONMENT	COMPLETED		NUMBER		UNDER CONSTRUCTION		NUMBER		APPROVED FOR CONSTRUCTION		NUMBER		BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Works Program Funds	Eliminated by Separation or Relocation	Protected By Signals or Otherwise	Estimated Total Cost	Works Program Funds	Eliminated by Separation or Relocation	Protected By Signals or Otherwise	Estimated Total Cost	Works Program Funds	Eliminated by Separation or Relocation	Protected By Signals or Otherwise	
Alabama	$ 4,034,617					$ 1,384,174	$ 1,384,174	17		$ 1,954,160	$ 1,954,160	16	1	$ 696,283
Arizona	1,256,099					50,000	50,000	1		623,661	469,908	5		736,191
Arkansas	3,574,060					535,740	534,897	13		730,904	728,256	21		2,310,908
California	7,486,362					3,530,358	3,363,315	22		2,499,737	2,428,900	13		1,694,148
Colorado	2,631,567					902,884	881,588	16		198,051	198,051	2		1,551,927
Connecticut	1,712,684													1,712,684
Delaware	418,239													418,239
Florida	2,827,883					713,695	713,695	6		554,303	554,303	6		1,559,885
Georgia	4,895,949									10,581	10,581			4,885,368
Idaho	1,674,479					542,501	518,887	9		288,974	248,811	3		906,781
Illinois	10,307,184					380,078	380,078	2		2,126,788	2,126,788	32		7,800,318
Indiana	5,111,096					1,671,256	1,671,256	9		1,411,865	1,411,864	8		2,027,976
Iowa	5,600,679					764,388	732,950	12		584,581	557,700	15		4,310,029
Kansas	5,246,258					78,694	78,694			2,242,473	2,242,473	23		2,925,091
Kentucky	3,672,387					165,603	140,290	2		2,438,067	1,910,539	15		1,621,558
Louisiana	3,213,467									1,293,840	1,026,000	13		2,187,467
Maine	1,426,861					141,606	141,272	3		240,550	240,550	4		1,045,038
Maryland	2,061,751									834,595	834,595	6		1,227,155
Massachusetts	4,210,833					756,997	756,997	3		317,872	317,872	1		3,135,965
Michigan	6,765,197					2,907,852	2,907,852	29		2,124,600	2,124,600	11		1,732,745
Minnesota	5,395,441	$ 48,769	$ 48,769	2		482,903	476,853	6		404,216	404,216	11	1	4,465,604
Mississippi	3,241,475					175,124	175,124	4		1,212,094	1,212,094	21		1,854,258
Missouri	6,142,153					232,638	232,638	3		1,237,380	1,237,380	10		4,672,135
Montana	2,722,327					1,441,742	1,441,742	26		495,590	495,590	10		784,995
Nebraska	3,556,441					712,177	712,177	25		1,113,231	1,113,231	30		1,731,033
Nevada	887,260					306,096	306,096	7		13,308	13,308			567,856
New Hampshire	822,484									222,531	222,531			599,953
New Jersey	3,983,826									462,237	462,237	2		3,521,589
New Mexico	1,725,286					316,721	302,895	5		530,164	507,919	4		914,472
New York	13,577,189					3,028,210	2,924,910	8		1,675,070	1,638,070	9		9,014,209
North Carolina	4,823,958					695,648	695,648	6		342,046	342,046	7		3,786,264
North Dakota	3,207,473					147,361	147,361	3		25,875	25,875	1		3,034,236
Ohio	8,439,897					126,598	126,598							8,313,299
Oklahoma	5,004,711					544,435	544,435	10		1,072,935	1,067,935	18		3,392,340
Oregon	2,334,204					778,822	777,061	5		718,416	718,416	5		838,727
Pennsylvania	11,483,613					219,670	203,527	5		1,108,400	1,063,900	18		10,216,186
Rhode Island	699,691					236,879	236,879			457,910	419,691	4		43,121
South Carolina	3,059,956					244,332	244,332	5		641,719	641,719	18		2,173,905
South Dakota	3,249,086					175,596	175,596	4		188,703	188,703	2		2,884,787
Tennessee	3,903,979					78,679	78,679	1		273,217	273,217	5		3,552,083
Texas	10,855,982					376,391	376,391	10		1,897,509	1,888,055	25		8,591,536
Utah	1,230,763					162,902	162,902	1		327,847	268,915	6		798,946
Vermont	729,857					351,069	349,711	4		138,704	138,704	3		241,442
Virginia	3,774,287					228,748	228,748	4		33,994	33,994	2		3,511,545
Washington	3,095,041					1,226,743	1,226,743	13		448,854	444,054	6		1,424,244
West Virginia	2,677,937													2,677,937
Wisconsin	5,022,683					984,727	921,991	10		1,283,987	1,283,987	9		2,816,705
Wyoming	1,360,841					55,366	55,365	2						1,305,476
District of Columbia	410,804									166,697	166,697	2		244,107
Hawaii	453,703									314,873	314,873	3		138,830
TOTALS	196,000,000	48,769	48,769	2		27,855,403	27,380,347	311		37,283,109	35,973,308	425	2	132,597,576

CURRENT STATUS OF UNITED STATES PUBLIC WORKS ROAD CONSTRUCTION

AS PROVIDED BY SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT (1934 FUNDS) AND BY THE ACT OF JUNE 18, 1934 (1935 FUNDS)

AS OF JANUARY 31, 1936

STATE	APPORTIONMENTS		COMPLETED				UNDER CONSTRUCTION				APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS	
	Sec. 204 of the Act of June 16, 1933 (1934 Fund)	Act of June 18, 1934 (1935 Fund)	Total Cost	1934 Public Works Funds	1935 Public Works Funds	Mileage	Estimated Total Cost	1934 Public Works Funds	1935 Public Works Funds	Mileage	1934 Public Works Funds	1935 Public Works Funds	Mileage	1934 Public Works Funds	1935 Public Works Funds
Alabama	$ 8,370,133	$ 4,259,842	$ 13,925,465	$ 8,131,070	$ 2,475,807	723.4	$ 1,229,130	$ 186,798	$ 1,042,332	49.2		$ 283,993	2.6	$ 52,264	$ 457,711
Arizona	5,211,960	2,641,935	8,402,449	5,190,976	2,199,301	527.6	411,729	4,927	334,060	17.5		34,394	.8	16,057	74,179
Arkansas	6,748,335	3,428,049	9,988,489	6,527,672	2,497,094	550.1	797,698	78,913	716,663	52.2		59,956	7.9	141,750	154,336
California	15,607,354	7,932,206	27,429,563	15,601,530	6,046,472	750.8	2,879,420		1,722,745	22.7		127,486	.2	5,823	35,504
Colorado	6,674,530	3,486,006	11,125,663	6,831,431	3,450,779	636.4	5,012		5,012			1,847	.1	43,099	28,368
Connecticut	2,865,740	1,454,868	4,182,222	2,825,079	941,105	69.3	359,166		343,645	3.7		58,859	.1	40,661	111,258
Delaware	1,819,088	923,395	2,645,565	1,819,088	775,652	128.3	32,528		32,528						115,214
Florida	5,231,834	2,661,343	8,236,102	5,211,895	1,835,538	281.7	633,343		551,388	25.6		175,525	1.7	19,939	98,892
Georgia	10,091,185	5,113,491	11,777,572	9,092,626	2,240,596	673.9	1,352,281	436,431	867,827	63.3	$ 82,205	770,247	45.7	479,923	1,234,821
Idaho	4,486,249	2,277,486	6,184,886	4,394,175	1,404,598	488.7	695,001		693,751	8.6		111,523	4.1	92,074	67,615
Illinois	17,570,770	8,921,401	20,868,437	15,568,270	4,765,941	580.7	5,483,515	1,920,565	3,562,950	114.1	3,838	338,505		78,098	254,005
Indiana	10,037,843	5,088,963	12,210,015	9,065,834	2,659,860	384.2	2,879,052	791,499	2,052,160	102.2	58,342	192,513	2.2	122,168	184,431
Iowa	10,055,660	5,118,361	15,413,603	9,939,405	4,698,510	1,220.3	297,801	114,154	162,100	4.9		1,000	2.3	2,101	256,751
Kansas	10,089,604	5,117,675	13,857,556	9,969,212	3,504,506	1,069.3	1,630,198	39,153	1,591,045	62.9	48,981	12,798	2.2	32,258	9,326
Kentucky	7,517,359	3,818,311	10,606,887	6,981,045	2,899,293	757.3	1,108,917	432,327	596,043	47.9	55,381	116,214	7.6	48,606	206,761
Louisiana	5,828,591	2,963,932	6,733,393	4,614,872	1,554,537	204.7	2,063,383	1,008,226	1,013,203	38.5	149,287	365,106	17.2	56,206	31,087
Maine	3,369,917	1,711,586	4,918,806	3,343,664	1,373,482	184.8	286,379		286,364	8.3				26,253	51,740
Maryland	3,564,527	1,810,058	4,296,510	2,786,846	600,931	113.3	1,163,202	706,935	450,722	31.3		162,409	4.9	70,745	595,996
Massachusetts	6,597,100	3,350,474	6,861,318	4,587,051	1,687,727	102.9	3,368,874	1,974,333	1,394,541	12.4				35,715	268,206
Michigan	12,736,227	6,452,568	17,999,163	12,544,952	4,468,553	673.1	2,097,802	117,227	1,922,598	95.7		30,825	.1	74,048	30,592
Minnesota	10,656,569	5,425,551	15,382,571	9,883,873	4,786,377	1,623.6	834,658	606,826	188,795	26.2	17,260	185,846	2.0	148,610	264,533
Mississippi	6,978,675	3,540,227	10,238,181	5,952,364	1,628,509	594.2	2,520,004	877,112	1,400,797	126.0	6,059	440,937	26.3	143,140	69,985
Missouri	12,180,306	6,173,740	14,928,766	10,868,487	3,105,538	1,350.1	4,224,185	1,185,562	2,765,861	100.6		248,256	.6	126,257	54,085
Montana	7,439,748	3,769,734	10,980,121	7,160,278	3,156,147	1,001.2	605,546	159,632	445,914	29.7		56,546		99,838	111,127
Nebraska	7,828,961	3,964,364	12,353,117	7,610,645	3,373,278	992.1	732,366	40,701	457,313	42.0		4,289		177,614	129,484
Nevada	4,545,917	2,302,356	6,650,573	4,502,450	1,991,464	735.6	351,978	26,150	289,722	21.7		4,675		17,317	16,494
New Hampshire	1,909,839	969,462	2,941,855	1,909,443	915,231	77.2	30,238		30,238	.5				397	23,993
New Jersey	6,346,039	3,220,879	6,948,192	6,002,237	587,836	74.9	2,636,867	175,941	2,143,993	12.4				167,861	489,050
New Mexico	5,792,935	2,941,700	8,209,865	5,631,430	2,375,529	747.6	439,280		439,280	19.7	114	777		161,391	126,114
New York	22,330,101	11,327,921	37,143,511	21,658,832	9,032,670	802.1	3,244,193	503,800	2,132,799	22.7	75,977	66,200	.9	91,492	96,251
North Carolina	9,522,293	4,840,941	14,019,596	8,813,464	3,989,583	1,322.0	910,370	320,633	572,842	21.6	65,953	212,599	4.3	322,243	65,917
North Dakota	5,804,448	2,938,967	7,308,213	5,510,239	1,151,028	1,949.5	753,940	130,778	564,567	112.3	67,289	537,809	62.1	96,143	685,562
Ohio	15,484,592	7,865,012	22,414,122	15,279,149	5,500,147	747.8	2,084,856	158,521	1,791,106	27.9		364,868	21.6	46,921	208,892
Oklahoma	9,216,798	4,685,180	12,899,467	9,041,780	2,987,813	761.8	1,431,422	153,088	1,276,599	42.5		183,352	1.9	21,931	237,416
Oregon	6,106,896	3,097,814	9,395,813	5,897,458	2,655,189	461.4	427,145	99,557	264,264	4.0		17,837	.8	109,880	160,524
Pennsylvania	18,891,004	9,590,788	27,240,779	17,884,029	8,131,300	982.4	1,597,179	738,565	772,568	76.8	5,126	307,671	2.2	263,284	379,249
Rhode Island	1,998,708	1,014,572	2,958,197	1,998,708	853,971	88.0	138,587		137,102	1.1		880			22,619
South Carolina	5,459,165	2,770,954	6,389,709	5,009,340	1,248,304	495.0	1,428,715	303,208	1,101,298	123.2	15,740	19,503	.2	130,878	401,849
South Dakota	6,011,479	3,047,643	8,528,768	5,779,245	2,200,439	1,471.1	413,519	15,396	398,123	50.3	62,646	299,804	24.2	154,192	149,277
Tennessee	8,492,619	4,302,991	11,966,612	8,182,177	2,915,872	460.9	1,285,310	293,634	895,275	22.9	13,049	185,008	2.0	3,760	306,835
Texas	24,244,024	12,291,353	32,931,119	23,379,310	7,997,100	2,617.8	5,081,355	834,209	4,018,207	157.8	1,537	156,661	16.7	28,969	119,285
Utah	4,194,708	2,132,691	6,912,098	4,156,356	1,862,147	584.0	327,904	6,060	267,191	6.9		1,500		32,291	1,853
Vermont	1,867,573	948,007	3,031,504	1,863,531	849,229	137.6	109,059	3,922	90,873	2.4		2,713		121	5,192
Virginia	7,416,757	3,765,387	11,169,043	7,180,354	3,073,047	568.9	505,354	99,518	398,492	48.7	42,221	46,881	.8	94,664	246,967
Washington	6,115,867	3,106,412	8,843,909	6,101,000	2,527,319	299.5	524,373		518,276	3.3		4,692		14,867	56,125
West Virginia	4,474,234	2,280,335	5,267,827	4,170,093	932,970	182.6	1,259,990	268,494	937,563	35.9	5,987			29,660	409,802
Wisconsin	9,724,881	4,941,837	14,855,100	9,642,681	4,318,541	604.7	572,508	7,300	522,766	15.3		93,112		74,900	7,418
Wyoming	4,501,327	2,287,712	6,537,332	4,433,878	1,923,598	987.8	359,722	46,705	286,281	49.5		47,239		20,744	30,595
District of Columbia	1,918,469	973,842	2,578,427	1,917,978	660,313	19.5	313,125		277,625	2.9				491	35,904
Hawaii	1,871,062	949,778	1,496,428	1,122,101		32.2	1,471,542	748,961	534,135	19.9		169,484	1.1		246,158
TOTALS	394,000,000	200,000,000	560,186,479	373,589,603	138,810,771	32,893.9	65,389,721	15,615,761	45,261,542	1,987.7	776,992	6,502,339	267.4	4,017,644	9,425,348